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DRAWINGS CITED IN THIS APPENDIX*

*The listed drawings are included as "General References" only; i.e., refer to the drawings to obtain additional detail or to obtain background information. These drawings are not part of the UFSAR. They are controlled by the Controlled Documents Program.

DRAWINGS*	SUBJECT
A-4	North Elevation
M-5	General Arrangement Roof Plan Units 1 & 2
M-14	General Arrangement Section "A-A" Units 1 & 2
M-20	General Arrangement River Screen House Units 1 & 2

CHAPTER 2.0 - SITE CHARACTERISTICS

2.1 GEOGRAPHY AND DEMOGRAPHY

2.1.1 Site Location and Description

2.1.1.1 Specification of Location

The Byron Station - Units 1 and 2 is located in northern Illinois, 3.7 miles south-southwest of the city of Byron, and 2.2 miles east of the Rock River, in Ogle County. The site is situated in the approximate center of the county in a predominately agricultural area. Figure 2.1-1 locates the site in relation to Illinois.

The site is located entirely within one political division, Rockvale Township. Site property occupies portions of sections 10, 11, 12, 13, 14, 15, and 24 (Township 24 North, Range 10 East). The location of the site within the county and within Rockvale Township is shown in Figures 2.1-2 and 2.1-3.

The station site is roughly rectangular in shape, with the plant structures occupying the southeast portion of the site. The approximate location of the reactors in Zone 16 of the Universal Transverse Mercator Coordinate System is given as follows.

UNIT	LATITUDE AND LONGITUDE	UTM COORDINATES
1	89° 16' 55" W x 42° 4' 29" N	4,661,800 N 310,700 E
2	89° 16' 55" W x 42° 4' 32" N	4,661,888 N 310,700 E

Byron is the closest sizeable city (1977 population 1884) to the site boundaries although Oregon is the largest city within a 10-mile radius (1977 population 3543). Figure 2.1-2 shows the major cities within Ogle County and the vicinity of the site.

Figure 2.1-4 indicates the major roads within the surrounding area. Illinois State Route 2 is the closest major highway to the site and is located 2.5 miles west of the Byron Station (centerline of the reactors). Other principal roads in the area are Illinois State Highways 72 and 64, located 3.5 miles north and 4.2 miles south of the station, respectively. The site itself is bounded by County Highway 2 (German Church Road) to the east, Deerpath Road to the south, and Razorville Road to the west. The closest railroad to the station is the Chicago, Milwaukee, St. Paul and Pacific. It is located 4.0 miles northeast, as indicated in Figure 2.1-5.

Figure 2.1-5a shows the route of the railroad spur connecting the site to the Chicago, Milwaukee, St. Paul and Pacific Railroad. From the edge of the site the spur extends approximately 3.6 miles on a 150 to 200-foot wide right-of-way that covers about 75 acres, most of which was formerly agricultural land. The spur connects to the now-abandoned Chicago and North Western Railroad tracks, which Commonwealth Edison purchased from that point for approximately 2.2 miles to the northwest to connect with the Chicago, Milwaukee, St. Paul and Pacific.

2.1.1.2 Site Area Map

The Byron Station occupies approximately 1782 acres of land. This area consists of the main site area and the transmission and pipeline corridor to the Rock River. The main site area occupies approximately 1398 acres, while the corridor occupies the remaining 384 acres. The exclusion area and site boundary lines are shown in Figure 2.1-6.

The topography of the main site area is indicated in Figure 2.1-6a. The topography in the northern half is dissected and slopes to the northeast. In the southern half, the land slopes to the southwest and is slightly dissected and rolling. A northwest-trending upland ridge extends through Section 13 and merges with a broader north-south ridge which parallels the site on the east. The elevations range from approximately 906 feet mean sea level (MSL) in the southeastern portion of the site to about 670 feet MSL in that portion of the pipeline corridor which lies adjacent to the Rock River. The grade level for the plant will be 869 feet MSL. Before construction, the northern half of the site was wooded with some cropland and the southern half was largely cropland.

The location and orientation of principal plant structures is shown in Figure 2.1-7.

There will be no industrial, institutional, commercial, recreational, or residential structures on the site other than those used by Exelon Generation Company in the normal conduct of its utility business. The development of the site for uses other than power generation and agriculture is not planned.

2.1.1.3 Boundaries for Establishing Effluent Release Limits

Title 10 of the Code of Federal Regulations Part 20.1302 requires that a licensee demonstrates by measurement or calculation that the total effective dose equivalent to the individual likely to receive the highest dose from the licensed operation does not exceed the annual dose limit.

Also 10 CFR 50.34a requires that "in the case of an application filed on or after January 2, 1971, the application shall also identify the design objectives and the means to be employed for keeping levels of radioactive material effluents to unrestricted areas as low as practicable."

The restricted area is used for establishing effluent release limits and enable the Licensee to fulfill the requirements of 10 CFR 20. The restricted area boundary is specified to be the site boundary, as shown in Figure 2.1-6. The expected concentrations of radionuclides in effluents are discussed in Section 5.2 and will be in compliance with 10 CFR 20.106 criteria. The minimum distances from the release point of gaseous effluents (the vent stack) to the nearest site boundary for each of the 16 directional segments are given in Table 2.1-la. The site boundary closest to the release point of gaseous effluents is in the south-southeast segment at a distance of 2625 feet. Liquid effluents will be discharged through the blowdown lines into the Rock River approximately 2.2 miles northwest of the reactors.

The plant exclusion area is located entirely within the site boundary lines. The minimum exclusion boundary distance from the gaseous release point for both reactors is 1375 feet. Distances from the release point of gaseous effluents to the exclusion area boundary for each of the 16 directional segments are given in Figure 2.1-6.

2.1.2 Exclusion Area Authority and Control

2.1.2.1 Authority

The Byron Station exclusion area as defined in 10 CFR 100 is located entirely within the site boundary, as shown in Figure 2.1-6. Since Exelon Generation Company owns all properties within the site boundary in fee simple, they also own the exclusion area and have the authority to determine and control all activities occurring within the exclusion area, including removal and exclusion of personnel or property from the site. The Licensee owns all mineral rights and easements for the exclusion area, as well as for the remainder of the site property.

For accident releases, the minimum exclusion area boundary distance is 445 meters, measured from the outer containment wall.

2.1.2.2 Control of Activities Unrelated to Plant Operation

Exelon Generation Company retains the authority to control any and all activities on the plant site, including the admission of visitors. Access is restricted by the use of security personnel and protective fencing.

Only employees of the Licensee or other authorized personnel are on the site. There is no one residing on state property. Using the site for purposes other than power generation (e.g., recreation) is not contemplated by the Licensee.

2.1.2.3 Arrangements for Traffic Control

Since the exclusion area is not traversed by any highway, railway, or waterway no traffic control arrangements have been considered.

2.1.2.4 Abandonment or Relocation of Roads

No public roads were closed or relocated due to the Byron Station.

2.1.3 Population Distribution

The population projections and the list of cities with their projected populations are generated by a system of Sargent & Lundy (S&L) developed computer programs (Reference 1). The demographic tables present the population figures broken into 16 directional segments and 10 distance increments surrounding the site, while the list of cities details major populations in the area, their distance and direction from the site, and their 2020 projected populations.

The U.S. Bureau of Census has established a system of population units known as enumeration districts. An enumeration district has no set size, shape, or population. The boundaries of the enumeration districts avoid crossing state and county lines, tracts, minor civil divisions, etc. The Department of the Army has established the geographic coordinates of the population centroid of each enumeration district. The centroid is the point which best represents the population concentration of the enumeration district. It is not necessarily the geographic center. The coordinates of the centroid, the population information, etc., are placed on a MEDLIST tape by the Department of the Army. The S&L system of computer programs uses these tapes as input. The present populations were determined directly from the MEDLIST tapes.

The population projections used to examine the regional demography were generated by a system of Sargent & Lundy (S&L) developed computer programs and are based on 1960 and 1970 census data. The population projections are presented on tables that are separated into sixteen 22.5° azimuthal segments centered on the centerline of the reactors and five distance increments surrounding the Byron Station site. The cities list enumerates the major population concentrations in the area, their distance and direction from the site, and their projected population. The 1970 populations were determined directly from the Department of the Army MEDLIST tapes (S&L 1974, revised 1977). The projected population distributions were made by a computer program using a modified "Ratio Technique," which is further described in Subsection 6.1.4.2 of the Byron Environmental Report.

The majority of the people, about 58%, who reside within 30 miles of the site live in urban areas such as the cities of Rockford and Belvidere, while the remainder (42%) live in rural communities. In Ogle County, the situation is reversed with only 39% of the county populace living in urban areas, and the remainder (61%) living on farms or in rural communities of 2500 people and less (Bureau of the Census 1981).

Projected future population distributions were made by a computer using a modified "ratio technique." The ratio technique essentially involves calculating the future population of an area by projecting the ratio of the total population of that area to the total population of a larger area containing the first, and for which population projections have already been made.

The ratio is projected with the basic assumption that for the first few years of the projection period the ratio changes at the rate calculated for a base period in the past, but after the first few years the rate of change in the ratio gradually decreases to zero, so that the ratio is constant after 20 years.

The basis of the ratio was that of township to state populations. The period from 1960 to 1970 was selected as the base period for the projections. Using the rate of change in the township-state ratio calculated for the 1960-1970 period, and the state population for the year in question (the state population was projected geometrically using the growth rate of the state population during the 1960-1970 period), the township population was projected. In order to incorporate the assumption that the township-state ratio does not continue to increase (or decrease) at the calculated rate indefinitely, the ratio was held constant after 20 years.

For greater accuracy in the 0- to 5-mile region, onsite house counts were conducted in 1972 and 1977. The number of houses counted was multiplied by three (the average number of people per household based on the Census Bureau's statistics for the area) to arrive at the population. Future populations for the 5-mile region, based on the 1977 house count, were also projected using the modified ratio technique.

2.1.3.1 Population Within 10 Miles

The population within 5 miles of the Byron site is shown in Table 2.1-1. These statistics are based on an onsite house count conducted during April 1977. The population per household was taken as three people, which is consistent with the Census Bureau statistics for the area and with the 1972 house count. Comparison of the 1972 and 1977 house count data indicates that the population within a 5-mile radius of the site is growing at a faster rate than was projected in 1972. The increased growth rate within the 5-mile area can be attributed to the large amount of new home construction which is occurring in the areas surrounding the cities of Byron and Oregon.

The overall demographic pattern for the area within 10 miles of the Byron Station, as determined from the 1977 population projections, is one of gradual growth but at a slightly slower rate than was projected in 1972. Table 2.1-1 shows the 1977 and projected population distribution for a 10-mile radius surrounding the Byron Station. Figure 2.1-8 identifies the cities located within this radius, as well as their distance and direction from the site.

The total 1977 population within 5 miles of the site is 6,885, which is projected to reach 10,694 by the year 2020. The average 1977 population density within this radius is 88 people per square mile, with the majority living between 4 and 5 miles from the site. This is due to the two cities located within 5 miles of the site: Byron, located 3.7 miles north-northeast of the site, and Oregon, located 5.1 miles south-southwest.

The total 1977 population within a 10-mile radius of the site is 20,821. S&L computer projections indicate that this area will gradually increase by a total of 10,795 people to reach 31,616 people by the year 2020. The average 1977 population density within this radius is 66 people per square mile. This is expected to increase to an average of 101 people per square mile by 2020.

The majority of the population within 10 miles lives between 5 and 10 miles from the site in the north-northeast, south-southwest, and west sectors. This is due primarily to the cities of: Byron, located 3.7 miles NNE; Oregon, located 5.1 miles SSW; and Mount Morris, located 7.5 miles WSW of the site.

2.1.3.2 Population Between 10 and 50 Miles

The 1970 and 1977 populations within 50 miles of the site and the projected populations for 10-year intervals until 2020 are shown in Table 2.1-2. The total 1977 population within 50 miles is 978,998 and is expected to increase by approximately 535,140 people to reach 1,514,138 by 2020.

The average 1977 population density is 125 people per square mile, which is expected to increase to an average of 193 people per square mile by the year 2020. The area between 10 and 20 miles from the site is the most densely populated with a 1977 population density of 269 people per square mile. The least densely populated area is within 10 miles of the site, with only 66 people per square mile.

The most heavily populated sectors within 50 miles of the site are the north-northeast and northeast sectors with 1977 populations of 210,510 and 197,489, respectively. The high populations in these sectors are due primarily to Rockford and its surrounding suburbs. Rockford is located approximately 17 miles northeast of the site and has a 1977 population of 160,575. The west-northwest sector, which is predominately rural, has the lowest 1977 population (13,687 people).

Figure 2.1-9 shows the location of the major cities within 50 miles of the site. All cities within this radius are listed in Table 2.1-3 which also gives their 1970 populations, their projected 2020 populations, and their distance and direction from the site.

2.1.3.3 Transient Population

The transient population within 10 miles of the site is composed primarily of visitors to local recreation areas and facilities, although some population movement and concentration can be attributed to schools and industries located within the 10-mile radius.

As shown in Table 2.1-4, there are three state parks within 10 miles of the site: Lowden Memorial State Park, located 3.5 miles southwest of the site; White Pines State Park, located 10.5 miles west-southwest of the site; and Castle Rock State Park, located 7 miles southwest of the site. In 1980, these three parks had a combined annual attendance of approximately 1,132,000 people (Illinois Department of Conservation, 1981). The estimated peak daily attendance for these areas are 6,800, 14,200, and 950 visitors, respectively.

Lowden Memorial State Park consists of 207 acres and offers boating, fishing, camping, picnicking, hiking, and summer interpretative program. Lowden State Park was visited by approximately 632,000 visitors in 1980. White Pines State Park consists of 385 acres and offers camping, fishing, picnicking, hiking, and summer interpretive program. Approximately 457,000 people visited the park in 1980. Castle Rock State Park will encompass approximately 1956 acres once land acquisition is completed, and surrounds a core area of 589 acres which forms the Castle Rock Nature Preserve. In 1980, 42,637 people visited the park, which is still undergoing development. When completed, the park will offer camping, boating, fishing, hiking, picnicking, and natural history interpretation.

In addition to these state recreational facilities, there are several privately-owned recreation areas within 10 miles of the Byron site. Table 2.1-4 lists the recreation areas within a 10-mile radius and indicates their size, location, and estimated annual and peak daily attendance. These recreation facilities are primarily family-oriented and offer a variety of activities for all ages. Because of the variety and availability of recreation facilities, many people from outside the 10-mile radius are attracted to this area during the recreational season (March to November). The majority of these visits occur during May through September, especially on weekends and holidays.

The estimated peak daily attendance figures in Table 2.1-4 indicate that on a short-term basis, the population within 10 miles of the site could increase by 43,617 people due to both state and private recreational facilities. Should all these people come from outside the 10-mile area surrounding the Byron site, which is highly unlikely, the total population of this area would be increased by 209%.

There are 28 industries within 10 miles of the site, as indicated in Table 2.1-10. Over 2,300 people are employed by these industries, or approximately 11% of the 1977 population located within 10 miles of the site. It can be assumed that most of these employees reside near their place of work, and are transient only in view of the fact that they do not work at home and travel to their place of business.

There are 16 schools within 10 miles of the site with a total 1980-1981 enrollment of 5066 and a staff of 426 teachers, as indicated in Table 2.1-6. The great majority of students attending these schools reside within a 10-mile radius of the site.

The 1977 and projected population within the 10-mile radius is given in Table 2.1-1. This table includes the residential and peak daily transient population resulting from recreational activities within the 10-mile area.

2.1.3.3.1 Land Use Within 5 Miles

The area within a 5-mile radius of the site is located entirely within Ogle County. Ogle County is predominantly agricultural with 91.8% of its total land acreage in farmland, according to the 1974 Agricultural Census (Bureau of the Census 1977a).

There are approximately 344,482 acres of farmland under cultivation in Ogle County. This area is approximately 71.0% of the total county acreage and 77.3% of the land devoted to agriculture. The major crops grown in Ogle County are corn and soybeans. Wheat, oats, and hay are also grown. In general, Ogle and Winnebago Counties produced less corn and soybeans and more wheat, oats, and hay than the state average. The one exception to this was county soybean production for 1975, which exceeded the state average. The acreage devoted to corn, wheat, oats, and hay either increased or remained the same between 1974 and 1975, while the acreage devoted to soybeans decreased. This was true for both Ogle and Winnebago Counties, and the State of Illinois. Corn and soybeans are the major crops grown within 5 miles of the site.

The major livestock raised in Ogle County are cattle and hogs. Table 2.1-10 lists livestock statistics for Ogle and Winnebago Counties for the years 1978 and 1979. A survey of milk cows and milk goats was made for the area within a 5-mile radius of the Byron Station. The results of the milk cow survey are shown in Table 2.1-11. One milk goat was found 4 miles south-southeast of the site (Huebner 1981), and there are several commercial dairy herds within the 5-mile radius, with sizes ranging from 50 to 300 head (Wm. S. Lawrence and Associates 1977). The milk produced in Ogle County is predominantly Grade A (60%) and is used for drinking. The major markets for Grade A milk are located in the Chicago and Rockford areas. The remainder of the milk produced is Grade B and is used in the processing of dairy products such as butter, cheese, and ice cream. Much of this milk is sent to Carroll, Jo Davies, and Stephenson Counties where the food processing plants are located (Smith 1977).

There are only two cities within 5 miles of the site, Byron and Oregon, which are located and identified in Figure 2.1-2 and discussed in Subsection 2.1.3.1. Residential land use is centered around these two cities and in a few outlying housing developments. The remainder of the 5-mile area is predominantly rural. Table 2.1-12 lists the nearest residence and garden to the site in each of the 16 sectors to a distance of 5 miles. The nearest residence and garden are located 0.7 mile from the site (Huebner 1981).

There are eight privately owned recreation areas, one county park, and one state park within 5 miles of the site. They are listed in Table 2.1-4 which also describes the areas in terms of size, location, facilities, and attendance. Lowden Memorial State Park is located 3.5 miles southwest of the site. Camping, picnicking, hiking, and natural history interpretation are available on 207 acres, while fishing and boating are permitted on the Rock River located adjacent to the park. Weld Memorial Park, located 3 miles east-northeast of the site, is a 35 acre park owned by Ogle County which offers camping, picnicking, hiking, and fishing.

There are eight schools within 5 miles of the site (Illinois State Board of Education, 1977). Table 2.1-6 lists all schools with their location, grade levels, staff, and enrollment. There are no hospitals within 5 miles of the site. The nearest hospital is located in Rochelle, Illinois, approximately 15 miles southeast of the site (American Hospital Association, 1972).

The area within 5 miles of the site is not heavily industrialized. The nearest industries are located in Byron, approximately 3.7 miles north-northeast. There are four industries in Byron and 17 in Oregon, as indicated in Table 2.1-7 which lists all industries within 10 miles of the site, their products, and their approximate employment. There are several quarries within 5 miles of the site (see Subsection 2.2.1.1).

There are two major oil pipelines and several small gas pipelines within 5 miles of the site. (Refer to Subsection 2.2.1.3 for further information.)

The major transportation routes within 5 miles of the Byron Station include highways and railroads. The Rock River, passing approximately 2.2 miles west of the plant, is used primarily for recreational purposes. The nearest road to the site is County Highway 2 (German Church Road), which traverses the site east of the exclusion area. Major highways located within 6 miles of the site include State Highways 2, 72, and 64, which are located and identified in Figure 2.1-4. Traffic volumes are also given in Figure 2.1-4. Illinois State Route 2, which is the closest major highway to the site, is located 2.5 miles west of the plant and has an annual average traffic flow per 24-hour period that ranges from 4,000 cars between Byron and Oregon to 8,800 cars in Oregon. State Routes 72 and 64 are also well traveled, having 24-hour annual averages that exceed 2,000 cars.

Figure 2.1-5 identifies and locates railroads within 6 miles of the site. There is one railroad within this area, the Chicago, Milwaukee, St. Paul, and Pacific, which passes through the city of Byron and is located 4.0 miles north of the station. The railroad operates no scheduled passenger trains through this area. Its primary purpose is freight transportation.

There are four airports within 5 miles of the site, three airstrips, and one seaplane base on the Rock River. All of the airports within 5 miles of the site are small private operations. (Refer to Subsections 2.2.1.5 and 2.2.2.5 for further information.)

The area within a 5-mile radius is zoned primarily as agricultural land, with residential, commercial, and industrial districts located in and near the cities of Byron and Oregon. There are several scattered recreational areas, some of which are privately owned and identified as commercial areas. Ogle County adopted a comprehensive land use plan in 1979 that encourages the continued use of the prime and good farmland in the county for agricultural production. Residential development is directed to and around the towns and villages of the county (Stevens 1981).

2.1.3.3.2 Surface Water

The most important body of water near the site is the Rock River. At its closest approach the Rock River is approximately 1.5 miles west of the western site boundary. This point is approximately 115 river miles upstream of the confluence of the Rock and Mississippi Rivers.

The Rock River is considered nonnavigable to commercial traffic except for that portion adjacent to the Mississippi River (U.S. Army Corps of Engineers, 1977). Only navigable waters fall under the U.S. Army Corps of Engineers permit procedures. The nearest dams to the site are located at Rockford, approximately 21.6 miles upstream of the intake point and one at Oregon, approximately 5.4 miles downstream from the intake point. The U.S. Army Corps of Engineers maintains no dams or locks on the Rock River, and it does not dredge the river channel.

The primary use of the Rock River is for recreation. Boating, fishing, and waterskiing are popular pastimes, and there are numerous residences located along the river banks. It is estimated, based on information provided by the site superintendent of Lowden Memorial State Park, that approximately 16,900 people use the river annually for recreation purposes between Byron and the Oregon Dam (Hayes 1977).

Sport fishing is a major recreational activity on the Rock River. Three creel surveys have been conducted near the Byron Station where the intake and discharge areas are situated: one conducted in 1967 by the Illinois Department of Conservation, one by Environmental Analysts Incorporated in 1972-1973, and one by Espey Huston & Associates in 1975. They also conducted additional creel surveys in 1976, 1977, 1978, and 1979; the data from these surveys are comparable to the data collected during the 1972 through 1975 surveys. During the 1972-1973 survey, a total of 954 interviews were conducted. Channel catfish, which were preferred by 47.4% of the fishermen interviewed, were the most abundant fish caught but carp accounted for the greatest weight. The average number of fish caught per rod-hour for the survey was 0.404. During the 1975 survey, 207 interviews were conducted. The most frequently caught species were carp (42.9%), channel catfish (26.7%), and shorthead redhorse (9.9%). Fifteen other species comprised the remaining 20.5% of the total catch. During the census it was determined that most fishermen caught less than one fish per hour of effort (0.197 fish caught per rod-hour), but that the average catch per trip was approximately 5.7 fish. It was estimated by 158 fishermen interviewed that they made approximately 12.4 trips per person annually to the Rock River (CECo 1977a).

Commercial fishing on the Rock River is limited to special contracts given by the Illinois Department of Conservation (CECo 1977a). There were only five commercial fishing operations, involving a total of 10 fishermen, registered by the Illinois Department of Conservation in 1976. Since 1972 the number of commercial fishermen has steadily declined. In 1976, 245,428 pounds of fish were taken. Buffalo and carp were the predominant fish caught, and buffalo accounted for the greatest weight. There is only one commercial fishing operation within Ogle County. It is owned by Mr. Lee Gibson, who fishes the river between the Dixon Dam and the Ogle County line (Illinois Department of Conservation 1977c).

The only other uses of the Rock River are for industrial water supplies and some irrigation. The Rock River and the tributaries are not used for public water supply (Purcell 1981). There are three industrial surface water intakes within 50 radial miles downstream of the site, all of which are located in either Dixon or Sterling. The closest industrial intake to the Byron site is the Lone Star Cement Company in Dixon. It withdraws approximately 0.25 cubic feet per second.

There is one farmer within Ogle County who uses the Rock River for irrigation. Rick McCanse irrigates 270 acres of corn at two different locations downstream from the site. The two fields are located approximately 4.7 and 6.7 miles from the intake and discharge points for the Byron Station. Rick McCanse uses approximately 8 or 9 acre inches of water per year and produces 170 bushels of corn per acre. The corn is sold to granaries located in Spring Valley and Hennepin, Illinois (Commonwealth Edison, 1977b).

2.1.3.3.3 Groundwater

Most of the water used for domestic, municipal, and industrial purposes within the general region of the site is obtained from wells. The municipal wells are supplied by either the Cambrian-Ordovician Aquifer or the Mt. Simon Aquifer, both of which are capable of high yields. Domestic uses are generally supplied by wells in the Galena-Platteville dolomites in the uppermost, less productive portion of the Cambrian-Ordovician Aquifer.

Within 10 miles of the site there are fifteen public water supplies. Five of these groundwater supplies provide water to municipalities; three serve Castle Rock State Park, Lowden Memorial State Park, and the Lorado Taft Field Campus of Northern Illinois University; and the remaining seven serve subdivisions and mobile home parks. (Refer to Subsection 2.4.13.2.1 for further information.)

There are 92 recorded wells located within 2.25 miles of the site and east of the Rock River. An additional 110 to 115 wells are located in an unincorporated area called Rock River Terrace, located approximately 2 miles northwest of the site. They are used primarily for agricultural and domestic purposes. (See Subsection 2.4.13.2.3.)

The operation of the Byron Station is not expected to significantly affect the Cambrian-Ordovician Aquifer or significantly change the groundwater levels or quality in domestic wells in the site area or in the nearest municipal wells. Some drawdown in neighboring wells, however, may occur due to the leakage across the Harmonoy Hill Shale member. This is explained more fully in Section 2.4. During plant construction, three privately owned wells located onsite were drilled for supplying batch plant and grouting operations. Two wells were drilled to supply the Byron Station with potable, sanitary, and demineralizer water, and makeup to the essential service water cooling towers.

A more detailed discussion of the hydrology of the Byron site area is presented in Section 2.4.

2.1.3.4 Low Population Zone

The low population zone (LPZ) as defined in 10 CFR 100 is "the area immediately surrounding the Exclusion Area which contains residents, the total number and density of which are such that there is a reasonable probability that appropriate protection measures could be taken in their behalf in the event of a serious accident." The 10 CFR 100.11 also lists numerical criteria to be met by the LPZ (for accidents analyzed using TID-14844), namely that the LPZ is "of such size that an individual located at any point on its outer boundary who is exposed to the radioactive cloud resulting from the postulated fission product release (during the entire period of passage) would not receive a total radiation dose to the whole body in excess of 25 rem or a total radiation dose in excess of 300 rem to the thyroid from iodine exposure." For accidents analyzed using Regulatory Guide 1.183 (AST), dose limits (in Rem TEDE) are listed in 10 CFR 50.67.

The low population zone that was chosen for the Byron Station consists of that area within a 3-mile radius (measured from the midpoint between the two reactors) of the site. The low population zone for the Byron Station is based on dose considerations as delineated in 10 CFR 100 and the population distribution around the station. The nearest population center, Rockford, meets the distance criteria set down in 10 CFR 100.11. Rockford is located 16.8 miles northeast of the site.

Figure 2.1-10 depicts the transportation routes and public facilities within the LPZ. The 1977 and projected population within the LPZ is given by sectors in Table 2.1-5. This table includes the residential population and the transient population resulting from activities in the LPZ.

As shown in Table 2.1-4, there are five recreation areas within 3 miles of the site, six including the Rock River. They are: Motosports Park, River Road Camping and Marina, the Mount Morris Boat Club, Weld Memorial Park, and the Byron Dragway. Motosports Park, located approximately 1 mile north of the site, is a public course for motorcycle and minibike riding. Races for these vehicles are also held there (Reference 4).

The River Road Camping and Marina has room for 100 camping vehicles, as well as facilities for boating, swimming, fishing, and waterskiing (Reference 5). It is located approximately 2 miles east of the site along the Rock River. The river itself is considered a recreational area because of the heavy recreational use it receives from the surrounding region. It is estimated that approximately 16,900 people use the stretch of the river near the plant site for boating, fishing, and waterskiing, during the recreational season (March-November).

The Mount Morris Boat Club is located approximately 2.5 miles northwest of the site, adjacent to the north bank of the Rock River. It is a small private club used primarily for boating activities.

Weld Memorial Park is a 35-acre recreation area with facilities for camping, fishing, hiking, and boating. There are also picnic tables and playground equipment available. Weld Memorial Park is located approximately 3 miles east-northeast of the site.

The Byron Dragway is a public recreational facility which is open for drag racing on the weekends, and sometimes for special 4-day racing meets. Approximately 1,000 to 12,000 people attend the dragway on the days when races are held (Reference 6). It is located 3 miles north of the site.

All of the previous recreational areas are open from March to November, although the busiest months are during the summer. Peak daily usage generally occurs on the weekends. Attendance figures are given in Table 2.1-4.

There are no schools within the LPZ, as indicated in Table 2.1-6. However, there are two schools located between 3 and 5 miles from the site with an enrollment of 1014 and a staff of 48. They are both located in Byron, 3.7 miles north-northeast of the site.

There are no major industries within the LPZ. Table 2.1-7 outlines the industries within 10 miles of the site, their employment, and products. There are also no known commercial establishments within the LPZ which could be expected to produce sizeable changes in the transient population of the area.

2.1.3.5 Population Center

A population center distance as defined in 10 CFR 100 means the distance from the reactor to the nearest boundary of a densely populated center containing more than 25,000 residents. Additionally, there must be "a population center distance of at least one and one-third times the distance from the reactor to the outer boundary of the low population zone." The closest such center is Rockford, Illinois. Its nearest boundary is located approximately 15 miles northeast of the reactors, although the city is actually located 17 miles northeast of the plant. Rockford had a 1970 population of 147,370, with an expected population of 160,575 by 1977, 166,181 by 1980, 202,979 by 2000, and 246,700 by 2020. Table 2.1-8 lists the population centers within 50 miles of the site, and Figure 2.1-11 locates them. There is a total of six population centers within a 50-mile radius.

Table 2.1-9 lists the distance and direction from the site and the 1970 population for all urban centers (population greater than 2500) within a 30-mile radius of the site. There are only 16 such centers of which three, Rockford, Freeport, and De Kalb, are population centers.

2.1.3.6 Population Density

The average population density within 50 miles of the site for 1980 is projected to be approximately 129 people per square mile. By 2020, the average density is projected to reach 193 people per square mile. Figure 2.1-12 shows the 1980 and 2020 projected populations with relation to uniform densities of 500 people per square mile and 1000 people per square mile for each of the 16 compass directions within 50 miles of the plant site. Tables 2.1-1 and 2.1-2 detail the cumulative populations shown in Figure 2.1-12.

2.1.4 References

1. DEMOG Computer Program, developed by Sargent & Lundy, July 1974, revised January 1977.

2. "Blackhawk Hills Project Plan," Blackhawk Hills Resource Conservation and Development Council, published by the Soil Conservation Service, U.S. Department of Agriculture, Champaign, Illinois, July 1975.

3. Illinois Department of Conservation, "Land & Historic Sites Attendance," December 1976.

4. Owner of Motosports Park, phone conversation with C. W. Comerford, S&L Cultural Resource Analyst, June 3, 1977.

5. Mr. Alvin Black, Manager of the River Road Camping and Marina, phone conversation with C. W. Comerford, S&L Cultural Resource Analyst, June 2, 1977.

6. Mr. Ron Leek, Owner of the Byron Dragway, phone conversation with C. W. Comerford, S&L Cultural Resource Analyst, August 29, 1977.

TABLE 2.1-1

1977 AND PROJECTED POPULATION DISTRIBUTION WITHIN 10 MILES OF THE BYRON STATION

SECTOR DESIGNATION	1977 RADIAL INTERVAL (mi)							
	0-1	1-2	2-3	3-4	4-5	0-5	5-10	0-10
N	0	3	15	70	697	785	399	1,184
NNE	3	3	34	1,021	670	1,731	700	2,431
NE	0	21	37	40	232	330	616	946
ENE	0	15	31	21	134	201	1,595	1,796
E	0	12	15	31	21	79	682	761
ESE	3	9	24	21	28	85	438	523
SE	0	18	12	15	43	88	189	277
SSE	6	24	6	37	18	91	669	760
S	3	6	15	15	147	186	396	582
SSW	21	15	6	40	1,163	1,245	1,453	2,698
SW	9	12	21	46	1,123	1,211	922	2,133
WSW	3	18	46	49	37	153	2,245	2,398
W	0	6	37	12	28	83	2,432	2,515
WNW	3	3	6	15	3	30	616	646
NW	0	205	266	15	34	520	565	1,085
NNW	0	9	9	21	28	67	19	86
Sum for Radial Interval	51	379	580	1,469	4,406	6,885	13,936	20,821
Cumulative Total to Outer Radius	51	430	1,010	2,479	6,885	6,885	20,821	20,821
Average Density (people/mi^2) in Radial Region	16	40	37	67	156	88	59	66

TABLE 2.1-1 (Cont'd)

SECTOR DESIGNATION	1980 RADIAL INTERVAL (mi) 0-1	1-2	2-3	3-4	4-5	0-5	5-10	0-10
N	0	3	16	75	719	813	410	1,223
NNE	3	3	37	1,059	691	1,793	691	2,484
NE	0	23	40	44	253	360	682	1,042
ENE	0	16	34	23	147	220	1,673	1,893
E	0	13	16	34	23	86	712	798
ESE	3	10	26	23	31	93	444	537
SE	0	20	13	16	44	93	192	285
SSE	6	26	7	39	18	96	685	781
S	3	6	17	15	148	189	399	588
SSW	22	16	6	40	1,166	1,250	1,474	2,724
SW	10	13	22	48	1,138	1,231	921	2,152
WSW	3	19	49	52	39	162	2,298	2,460
W	0	6	39	13	30	88	2,493	2,581
WNW	3	3	6	16	3	31	636	667
NW	0	217	281	16	36	550	561	1,111
NNW	0	10	10	22	29	71	19	90
Sum for Radial Interval	53	404	619	1,535	4,515	7,126	14,290	21,416
Cumulative Total to Outer Radius	53	457	1,076	2,611	7,126	7,126	21,416	21,416
Average Density (people/mi^2) in Radial Region	17	43	39	70	160	91	61	68

TABLE 2.1-1 (Cont'd)

SECTOR DESIGNATION	1990 RADIAL INTERVAL (mi) 0-1	1-2	2-3	3-4	4-5	0-5	5-10	0-10
N	0	4	18	87	796	905	446	1,351
NNE	4	4	45	1,181	766	2,000	648	2,648
NE	0	28	49	53	306	436	857	1,293
ENE	0	20	41	28	178	267	1,867	2,134
E	0	16	20	41	28	105	790	895
ESE	4	12	32	28	37	113	476	589
SE	0	24	16	20	49	109	206	315
SSE	8	31	8	45	20	112	748	860
S	4	7	19	16	159	205	432	637
SSW	26	18	7	43	1,231	1,325	1,621	2,946
SW	11	15	26	55	1,223	1,330	997	2,327
WSW	4	22	56	60	45	187	2,515	2,702
W	0	7	45	15	34	101	2,735	2,836
WNW	4	4	7	18	4	37	700	737
NW	0	250	324	18	40	632	587	1,219
NNW	0	11	11	24	32	78	23	101
Sum for Radial Interval	65	473	724	1,732	4,948	7,942	15,648	23,590
Cumulative Total to Outer Radius	65	538	1,262	2,994	7,942	7,942	23,590	23,590
Average Density (people/mi^2) in Radial Region	21	50	46	79	175	101	66	75

TABLE 2.1-1 (Cont'd)

SECTOR DESIGNATION	2000 RADIAL INTERVAL (mi)							
	0-1	1-2	2-3	3-4	4-5	0-5	5-10	0-10
N	0	4	20	98	878	1,000	490	1,490
NNE	4	4	51	1,306	844	2,209	607	2,816
NE	0	31	55	60	347	493	1,034	1,527
ENE	0	22	46	31	202	301	2,052	2,353
E	0	18	22	46	31	117	870	987
ESE	4	13	36	31	42	126	524	650
SE	0	27	18	22	54	121	226	347
SSE	8	34	9	49	22	122	826	948
S	4	8	21	18	174	225	478	703
SSW	29	20	8	47	1,341	1,445	1,804	3,249
SW	12	16	29	61	1,339	1,457	1,109	2,566
WSW	4	24	63	67	50	208	2,772	2,980
W	0	8	50	16	38	112	3,015	3,127
WNW	4	4	8	20	4	40	773	813
NW	0	279	362	20	43	704	640	1,344
NNW	0	12	12	26	35	85	26	111
Sum for Radial Interval	69	524	810	1,918	5,444	8,765	17,246	26,011
Cumulative Total to Outer Radius	69	593	1,403	3,321	8,765	8,765	26,011	26,011
Average Density (people/mi^2) in Radial Region	22	56	52	87	193	112	73	83

TABLE 2.1-1 (Cont'd)

2010

SECTOR DESIGNATION	RADIAL INTERVAL (mi) 0-1	1-2	2-3	3-4	4-5	0-5	5-10	0-10
N	0	5	22	109	971	1,107	536	1,643
NNE	5	5	57	1,445	934	2,446	557	3,003
NE	0	35	62	67	387	551	1,236	1,787
ENE	0	25	52	35	225	337	2,258	2,595
E	0	20	25	52	35	132	956	1,088
ESE	5	15	40	35	47	142	574	716
SE	0	30	15	25	60	130	253	383
SSE	10	39	10	55	25	139	906	1,045
S	5	9	24	19	191	248	527	775
SSW	32	23	9	52	1,469	1,585	1,997	3,582
SW	14	18	32	68	1,470	1,602	1,227	2,829
WSW	5	27	69	74	56	231	3,054	3,285
W	0	9	56	18	42	125	3,322	3,447
WNW	5	5	9	23	5	47	849	896
NW	0	309	401	22	49	781	700	1,481
NNW	0	14	14	29	39	96	27	123
Sum for Radial Interval	81	588	897	2,128	6,005	9,699	18,979	28,678
Cumulative Total to Outer Radius	81	669	1,566	3,694	9,699	9,699	28,678	28,678
Average Density (people/mi^2) in Radial Region	26	62	57	97	212	124	81	91

TABLE 2.1-1 (Cont'd)

SECTOR DESIGNATION	2020 RADIAL INTERVAL (mi) 0-1	1-2	2-3	3-4	4-5	0-5	5-10	0-10
N	0	5	24	121	1,069	1,219	592	1,811
NNE	6	6	63	1,592	1,029	2,696	513	3,209
NE	0	39	69	74	431	613	1,460	2,073
ENE	0	28	58	39	249	374	2,487	2,861
E	0	22	28	58	39	147	1,052	1,199
ESE	6	17	45	39	52	159	631	790
SE	0	33	22	28	66	149	273	422
SSE	11	43	11	61	27	153	999	1,152
S	5	10	26	22	209	272	582	854
SSW	35	25	10	57	1,611	1,738	2,210	3,948
SW	15	20	35	75	1,616	1,761	1,357	3,118
WSW	5	30	77	82	62	256	3,366	3,622
W	0	10	62	20	47	139	3,662	3,801
WNW	5	5	10	25	5	50	938	988
NW	0	342	444	24	53	863	770	1,633
NNW	0	15	15	32	43	105	30	135
Sum for Radial Interval	88	650	999	2,349	6,608	10,694	20,922	31,616
Cumulative Total to Outer Radius	88	738	1,737	4,086	10,694	10,694	31,616	31,616
Average Density (people/mi^2) in Radial Region	28	69	64	107	234	136	89	101

TABLE 2.1-1a

DISTANCE FROM GASEOUS EFFLUENT RELEASE POINT TO NEAREST SITE BOUNDARY IN THE 16 MAJOR COMPASS DIRECTIONS

DIRECTION	APPROXIMATE DISTANCE (ft)
N	6150
NNE	6000
NE	5200
ENE	4050
E	4025
ESE	3250
SE	3300
SSE	2625
S	3100
SSW	3200
SW	3500
WSW	3975
W	3900
WNW	4025
NW	3700
NNW	3425

TABLE 2.1-2

1970 AND PROJECTED POPULATION DISTRIBUTION
WITHIN 10-50 MILES OF THE BYRON STATION

SECTOR DESIGNATION	1970 RADIAL INTERVAL (mi) 10-20	20-30	30-40	40-50	0-50
N	3,030	3,358	8,011	6,923	22,419
NNE	51,034	24,787	55,782	55,335	189,243
NE	125,879	29,156	5,861	16,940	178,536
ENE	4,232	17,945	8,233	27,937	59,889
E	1,463	5,904	11,315	36,175	55,524
ESE	1,497	31,918	15,130	15,947	64,987
SE	11,253	2,057	4,258	24,791	42,616
SSE	1,430	1,740	10,550	4,031	18,467
S	2,572	4,141	3,219	10,580	21,088
SSW	9,647	4,081	3,259	5,541	25,213
SW	9,098	29,986	11,620	6,781	59,611
WSW	3,691	3,291	5,837	35,839	50,928
W	1,435	3,049	5,114	9,778	21,752
WNW	2,393	3,515	2,314	4,692	13,513
NW	1,289	32,880	5,523	5,397	46,120
NNW	3,053	3,264	7,118	9,482	22,991
Sum for Radial Interval	232,996	201,072	163,144	276,169	892,897
Cumulative Total to Outer Radius	252,512	453,584	616,728	892,897	892,897
Average Density (people/mi^2) in Radial Region	247	128	74	98	114

TABLE 2.1-2 (Cont'd)

SECTOR DESIGNATION	1977 RADIAL INTERVAL (mi) 10-20	20-30	30-40	40-50	0-50
N	3,327	3,641	8,702	7,042	23,896
NNE	55,280	29,028	58,759	65,012	210,510
NE	138,350	33,617	6,251	18,325	197,489
ENE	5,514	21,075	9,220	32,219	69,824
E	1,563	6,545	12,607	40,846	62,322
ESE	1,588	41,383	18,177	19,080	80,751
SE	13,365	2,003	4,307	28,911	48,863
SSE	1,420	1,816	11,150	4,166	19,312
S	2,680	4,229	3,164	11,053	21,708
SSW	9,218	3,874	3,191	5,629	24,610
SW	8,763	30,654	12,142	6,916	60,608
WSW	3,632	3,263	6,073	36,770	52,136
W	1,391	2,976	5,007	9,734	21,623
WNW	2,444	3,730	2,323	4,544	13,687
NW	1,243	34,440	5,611	5,256	47,635
NNW	3,205	3,472	7,379	9,882	24,024
Sum for Radial Interval	252,983	225,746	174,063	305,385	978,998
Cumulative Total to Outer Radius	273,804	499,550	673,613	978,998	978,998
Average Density (people/mi^2) in Radial Region	269	144	79	108	125

TABLE 2.1-2 (Cont'd)

SECTOR DESIGNATION	1980 RADIAL INTERVAL (mi)				
	10-20	20-30	30-40	40-50	0-50
N	3,451	3,764	9,009	7,150	24,597
NNE	57,133	30,638	60,295	68,794	219,344
NE	143,554	35,342	6,436	18,947	205,321
ENE	5,974	22,267	9,616	33,873	73,623
E	1,609	6,808	13,132	42,701	65,048
ESE	1,630	44,781	19,314	20,254	86,516
SE	14,159	2,005	4,360	30,486	51,295
SSE	1,429	1,857	11,440	4,245	19,752
S	2,739	4,295	3,176	11,300	22,098
SSW	9,160	3,842	3,199	5,705	24,630
SW	8,731	31,137	12,417	7,021	61,458
WSW	3,644	3,284	6,202	37,224	52,814
W	1,389	2,980	5,016	9,820	21,786
WNW	2,483	3,832	2,347	4,536	13,865
NW	1,239	35,243	5,688	5,263	48,544
NNW	3,281	3,570	7,534	10,112	24,587
Sum for Radial Interval	261,605	235,645	179,181	317,431	1,015,278
Cumulative Total to Outer Radius	283,021	518,666	697,847	1,015,278	1,015,278
Average Density (people/mi^2) in Radial Region	278	150	82	112	129

TABLE 2.1-2 (Cont'd)

SECTOR DESIGNATION	1990 RADIAL INTERVAL (mi) 10-20	20-30	30-40	40-50	0-50
N	3,834	4,166	10,087	7,850	27,288
NNE	63,215	35,499	66,825	78,648	245,935
NE	159,469	39,754	7,105	21,118	228,739
ENE	6,909	25,158	10,736	38,099	83,036
E	1,773	7,579	14,643	47,778	72,668
ESE	1,793	51,721	21,940	22,990	99,033
SE	16,040	2,147	4,726	34,390	57,618
SSE	1,541	2,034	12,569	4,634	21,638
S	2,997	4,671	3,408	12,373	24,086
SSW	9,736	4,073	3,431	6,192	26,378
SW	9,308	33,858	13,595	7,631	66,719
WSW	3,914	3,537	6,781	38,884	55,818
W	1,484	3,193	5,381	10,484	23,378
WNW	2,699	4,216	2,538	4,846	15,036
NW	1,322	38,616	6,173	5,663	52,993
NNW	3,598	3,931	8,312	11,184	27,126
Sum for Radial Interval	289,632	263,253	198,250	352,764	1,127,489
Cumulative Total to Outer Radius	313,222	576,475	774,725	1,127,489	1,127,489
Average Density (people/mi^2) in Radial Region	307	168	90	125	144

TABLE 2.1-2 (Cont'd)

SECTOR DESIGNATION	2000 RADIAL INTERVAL (mi) 10-20	20-30	30-40	40-50	0-50
N	4,226	4,591	11,278	8,776	30,361
NNE	69,696	38,154	74,597	87,927	273,190
NE	175,811	43,830	7,865	23,514	252,547
ENE	7,617	27,736	11,837	42,002	91,545
E	1,955	8,357	16,144	52,672	80,115
ESE	1,977	57,026	24,191	25,345	109,189
SE	17,686	2,368	5,210	37,917	63,528
SSE	1,700	2,243	13,858	5,107	23,856
S	3,305	5,148	3,760	13,641	26,557
SSW	10,735	4,491	3,783	6,827	29,085
SW	10,262	37,331	14,987	8,414	73,560
WSW	4,316	3,900	7,475	40,689	59,360
W	1,636	3,520	5,931	11,369	25,583
WNW	2,975	4,647	2,798	5,344	16,577
NW	1,457	42,570	6,808	6,289	58,468
NNW	3,966	4,335	9,283	12,505	30,200
Sum for Radial Interval	319,320	290,247	219,805	388,338	1,243,721
Cumulative Total to Outer Radius	345,331	635,578	855,383	1,243,721	1,243,721
Average Density (people/mi^2) in Radial Region	339	185	100	137	158

TABLE 2.1-2 (Cont'd)

SECTOR DESIGNATION	2010 RADIAL INTERVAL (mi) 10-20	20-30	30-40	40-50	0-50
N	4,659	5,062	12,608	9,813	33,785
NNE	76,842	42,072	83,270	98,307	303,494
NE	193,844	48,326	8,701	26,183	278,841
ENE	8,397	30,577	13,050	46,309	100,928
E	2,155	9,214	17,797	58,077	88,331
ESE	2,180	62,871	26,672	27,945	120,384
SE	19,497	2,609	5,743	41,802	70,034
SSE	1,873	2,473	15,278	5,631	26,300
S	3,644	5,678	4,146	15,040	29,283
SSW	11,836	4,953	4,171	7,526	32,068
SW	11,315	41,154	16,523	9,277	81,098
WSW	4,758	4,300	8,243	42,629	63,215
W	1,804	3,882	6,541	12,341	28,015
WNW	3,281	5,123	3,085	5,892	18,277
NW	1,608	46,936	7,506	6,983	64,514
NNW	4,373	4,778	10,368	13,982	33,624
Sum for Radial Interval	352,066	320,008	243,702	427,737	1,372,191
Cumulative Total to Outer Radius	380,744	700,752	944,454	1,372,191	1,372,191
Average Density (people/mi^2) in Radial Region	374	204	111	151	175

TABLE 2.1-2 (Cont'd)

SECTOR DESIGNATION	2020 RADIAL INTERVAL (mi)				
	10-20	20-30	30-40	40-50	0-50
N	5,137	5,583	14,097	10,970	37,598
NNE	84,721	46,388	92,955	109,909	377,182
NE	213,715	53,284	9,628	29,153	307,853
ENE	9,258	33,713	14,386	51,054	111,272
E	2,376	10,157	19,621	64,028	97,381
ESE	2,403	69,315	29,405	30,811	132,724
SE	21,495	2,877	6,332	46,088	77,214
SSE	2,066	2,726	16,843	6,209	28,996
S	4,017	6,259	4,571	16,583	32,284
SSW	13,049	5,460	4,599	8,299	35,355
SW	12,475	45,376	18,220	10,228	89,417
WSW	5,247	4,740	9,088	44,712	67,409
W	1,989	4,278	7,209	13,407	30,684
WNW	3,617	5,650	3,402	6,496	20,153
NW	1,773	51,748	8,275	7,753	71,182
NNW	4,821	5,269	11,577	15,632	37,434
Sum for Radial Interval	388,159	352,823	270,208	471,332	1,514,138
Cumulative Total to Outer Radius	419,775	772,598	1,042,806	1,514,138	1,514,138
Average Density (people/mi^2) in Radial Region	412	225	123	167	193

TABLE 2.1-3

CITIES WITHIN 50-MILE RADIUS OF BYRON STATION

CITY*	1970 POPULATION	2020 POPULATION	DISTANCE AND DIRECTION FROM THE SITE
Byron	1,749	2,877	3.7 miles NNE
Oregon	3,539	5,174	5.1 miles SSW
Stillman Valley	871	1,785	5.7 miles ENE
Leak River	633	997	6.5 miles WNW
Mount Morris	3,173	4,989	7.5 miles WSW
Adeline	156	229	11.1 miles WNW
Winnebago	1,285	2,235	13.0 miles N
Morristown	669	1,107	13.1 miles ENE
German Valley	206	300	13.3 miles NW
Hillcrest	630	1,213	13.9 miles SE
Ashton	1,112	1,657	14.8 miles SSE
Forreston	1,227	1,898	14.9 miles WNW
Ken Rock (U)**	5,945	10,095	15.0 miles NE
West End (U)**	7,554	12,505	15.2 miles NNE
Rochelle	8,594	16,546	15.3 miles SE
Polo	2,542	3,616	15.8 miles WSW
Franklin Grove	968	1,609	16.2 miles S
Pecatonica	1,781	2,895	16.3 miles NNW
Rockford	147,370	246,700	16.8 miles NE
Ridott	244	355	17.6 miles NNW

* Cities in Illinois unless otherwise specified.
** (U) indicates an unincorporated area.

TABLE 2.1-3 (Cont'd)

CITY*	1970 POPULATION	2020 POPULATION	DISTANCE AND DIRECTION FROM THE SITE
Dixon	18,147	24,091	18.6 miles SSW
Creston	595	975	19.1 miles ESE
Cherry Valley	952	2,586	19.8 miles ENE
Loves Park	12,390	21,656	20.2 miles NE
Steward	308	447	20.7 miles SE
North Park (U)**	15,679	32,445	21.5 miles NE
Kirkland	1,138	1,953	22.1 miles E
Freeport	27,736	42,530	22.3 miles NW
Shannon	848	1,337	23.1 miles WNW
Malta	961	1,572	23.6 miles ESE
Davis	525	849	24.2 miles NNW
Dakota	440	782	24.2 miles NNW
Durand	972	1,620	24.3 miles N
Rock City	251	406	24.4 miles NNW
Amboy	2,184	3,371	24.9 miles S
Nelson	263	470	24.9 miles SW
Belvidere	14,061	26,650	25.3 miles ENE
Milledgeville	1,130	1,580	25.5 miles WSW
Lee	252	362	25.8 miles SE
Cedarville	578	973	26.6 miles NW
Kingston	481	806	26.7 miles E

* Cities in Illinois unless otherwise specified.
** (U) indicates an unincorporated area.

TABLE 2.1-3 (Cont'd)

CITY*	1970 POPULATION	2020 POPULATION	DISTANCE AND DIRECTION FROM THE SITE
West Brooklyn	225	402	27.3 miles SSE
Lanark	1,495	2,155	27.3 miles W
Sterling	16,113	24,457	27.7 miles SW
Harmon	205	258	27.9 miles SSW
Rockton	2,099	3,233	27.9 miles NNE
Compton	399	713	28.1 miles SSE
De Kalb	32,949	73,346	28.6 miles ESE
Rock Falls	10,287	14,897	28.9 miles SW
Sterling West (U)**	2,171	3,295	29.3 miles SW
Pearl City	535	877	29.7 miles WNW
Shabbona	730	1,075	29.8 miles SE
Sublette	361	523	29.8 miles S
Genoa	3,003	5,532	29.9 miles E
South Beloit	3,804	6,176	30.4 miles NNE
Chadwick	605	813	30.4 miles W
Poplar Grove	607	1,176	30.6 miles NE
Paw Paw	846	1,305	30.7 miles SSE
Sycamore	7,843	13,741	30.7 miles ESE
Orangeville	538	814	31.8 miles NW
Cortland	541	993	31.9 miles ESE
Beloit West, (U)** Wisconsin	1,903	3,249	32.0 miles NNE

* Cities in Illinois unless otherwise specified.
** (U) indicates an unincorporated area.

TABLE 2.1-3 (Cont'd)

CITY*	1970 POPULATION	2020 POPULATION	DISTANCE AND DIRECTION FROM THE SITE
Beloit, Wisconsin	35,729	60,664	32.1 miles NNE
Waterman	990	1,539	33.3 miles SE
Lena	1,691	2,568	33.5 miles NW
Perry Go Place, (U)** Wisconsin	5,912	10,095	34.0 miles NNE
Mount Carroll	2,143	3,135	34.5 miles W
Capron	654	1,031	34.9 miles NE
Marengo	4,235	7,374	36.2 miles ESE
Ohio	506	673	36.6 miles SSW
Mendota	6,902	11,201	36.8 miles SSE
Burlington	456	762	37.1 miles E
Brodhead, Wisconsin	2,515	4,012	37.3 miles N
La Moille	669	975	37.4 miles S
Orfordville, Wisconsin	888	1,803	37.5 miles N
Deer Grove	66	84	37.6 miles SSW
Maple Park	660	1,007	37.7 miles ESE
Earlville	1,410	2,272	37.9 miles SSE
Winslow	330	410	37.9 miles NW
Hampshire	1,611	2,775	38.0 miles E
Morrison	4,387	7,050	38.6 miles WSW

* Cities in Illinois unless otherwise specified.
** (U) indicates an unincorporated area.

TABLE 2.1-3 (Cont'd)

CITY*	1970 POPULATION	2020 POPULATION	DISTANCE AND DIRECTION FROM THE SITE
Walnut	1,295	1,951	38.6 miles SSW
Hinckley	1,053	1,741	38.7 miles ESE
Union	579	943	38.7 miles ENE
Clinton, Wisconsin	1,333	2,168	38.7 miles NNE
Tampico	838	1,245	39.3 miles SW
Monroe, Wisconsin	8,654	14,509	39.3 miles NNW
Sharon, Wisconsin	1,216	1,968	40.0 miles NE
Leland	743	1,184	40.1 miles SE
Lyndon	673	981	40.3 miles SW
Footville, Wisconsin	698	1,120	40.5 miles N
Stockton	1,930	2,940	40.5 miles WNW
Harvard	5,177	8,500	40.9 miles NE
Arlington	250	365	41.4 miles S
Thomson	617	1,037	41.5 miles WSW
Nora	175	209	41.8 miles NW
Browntown, Wisconsin	253	370	42.2 miles NW
Prophetstown	1,915	2,946	42.6 miles SW
Savanna	4,942	6,941	42.7 miles W
Somonauk	1,112	1,830	42.9 miles SE
Coleta	208	323	42.9 miles SW

* Cities in Illinois unless otherwise specified.

TABLE 2.1-3 (Cont'd)

CITY*	1970 POPULATION	2020 POPULATION	DISTANCE AND DIRECTION FROM THE SITE
Troy Grove	281	506	43.1 miles SSE
Janesville, Wisconsin	46,426	93,500	43.1 miles NNE
Elburn	1,122	2,084	43.5 miles ESE
Huntley	1,432	2,603	43.6 miles E
Albany, Wisconsin	875	1,314	43.7 miles N
Pingree Grove	174	293	43.8 miles E
South Wayne, Wisconsin	436	831	44.0 miles NW
New Bedford	152	181	44.2 miles SSW
Dover	176	257	44.3 miles S
Sabula, Iowa	845	929	44.4 miles W
Sandwich	5,056	9,132	44.5 miles SE
Cherry	551	804	44.5 miles S
Malden	262	376	44.6 miles S
Woodstock	10,226	17,718	45.1 miles ENE
Warren	1,523	2,272	45.2 miles NW
Gilberts	336	580	45.7 miles E
Fulton	3,630	5,783	45.8 miles WSW
Darien, Wisconsin	839	1,358	46.0 miles NE
Walworth, Wisconsin	1,637	2,800	46.4 miles NE
Manlius	402	544	46.5 miles SSW

* Cities in Illinois unless otherwise specified.

TABLE 2.1-3 (Cont'd)

CITY*	1970 POPULATION	2020 POPULATION	DISTANCE AND DIRECTION FROM THE SITE
Plano	4,664	9,126	47.0 miles SE
Ladd	1,328	2,034	47.3 miles S
Sugar Grove	1,230	2,798	47.4 miles ESE
Clinton, Iowa	34,719	38,994	47.6 miles WSW
Lakewood	782	1,484	47.7 miles ENE
Monticello, Wisconsin	870	1,498	47.9 miles NNW
Fontana on Geneva, Wisconsin	1,464	2,524	47.9 miles NE
Evansville, Wisconsin	2,992	4,870	48.0 miles N
Andover, Iowa	90	100	48.2 miles W
Sheridan	724	1,015	48.4 miles SE
Princeton	6,959	11,670	48.5 miles S
Hollowayville	94	152	48.6 miles S
Seatonville	318	489	48.7 miles S
Wonder Lake (U)**	4,806	6,507	48.8 miles ENE
Lake in the Hills	3,240	6,182	48.8 miles E
Erie	1,566	1,877	48.9 miles SW
Crystal Lake	14,541	24,462	48.9 miles ENE
Sleepy Hollow	1,729	2,857	49.1 miles E
Hooppole	227	404	49.3 miles SW
Dalzell	579	887	49.3 miles S

* Cities in Illinois unless otherwise specified.
** (U) indicates an unincorporated area.

TABLE 2.1-3 (Cont'd)

CITY*	1970 POPULATION	2020 POPULATION	DISTANCE AND DIRECTION FROM THE SITE
Millington	338	505	49.4 miles SE
Elizabeth	707	998	49.5 miles WNW
Gratiot, Wisconsin	249	366	49.6 miles NW
Delavan, Wisconsin	5,526	6,260	49.6 miles NE
Apple River	482	807	49.8 miles NW
South Elgin	4,289	7,428	49.9 miles E

* Cities in Illinois unless otherwise specified.

TABLE 2.1-4

MAJOR RECREATIONAL AREAS WITHIN 10 MILES OF THE BYRON STATION

RECREATIONAL AREA	ACREAGE	DISTANCE & DIRECTION FROM SITE	ESTIMATED ANNUAL ATTENDANCE (1980)	ESTIMATED PEAK DAY ATTENDANCE
Moto Sports Park	50	1 mile N	50,000	5,000
River Road Camping & Marina	30	2 miles E	3,500	600
Rock River	NA	2.2 miles E	NA[a]	NA
Mt. Morris Boat Club	5	2.5 miles NW	NA	150
Weld Memorial Park	35	3 miles ENE	2,500	900
Byron Dragway	190	3 mile N	80,000	12,000
The Stronghold	460	3.5 miles WSW	19,100	350
Lowden Memorial State Park	207	3.5 miles SW	632,148	6,808
Lake Louise	52	4.5 miles NNE	NA	360
Oregon Country Club	94	4.5 miles SSW	NA	200
Camp Medill McCormick	300	6.5 miles NNE	NA	NA
Castle Rock State Park & Nature Preserve	1,956	7.0 miles SW	42,637	968
White Pines Ranch	100	8.0 miles WSW	29,600	250
Fuller Memorial Forest Preserve	NA	9.5 miles NE	12,000	600
Hansens-Hideaway	160	9.8 miles WSW	NA	NA

[a]NA - not available.

TABLE 2.1-4 (Cont'd)

RECREATIONAL AREA	ACREAGE	DISTANCE & DIRECTION FROM SITE	ESTIMATED ANNUAL ATTENDANCE (1980)	ESTIMATED PEAK DAY ATTENDANCE
White Pines State Park	385	10.5 miles WSW	457,472	14,176
Lake LaDonna	28	10.5 miles WSW	18,000	1,200
Camp Lowden	230	10.5 miles SW	2,000	200

Sources: Anderson (1981); Bent (1981); Collins (1981); Gaston (1981); Glotfelty (1981); Keister (1981); Leak (1981); Lihle (1981); Overton (1981); Richardson (1981); Smith (1981); Van Meter (1981); Vincar (1981); Etnyre (1982).

[a]NA - not available.

TABLE 2.1-5

1977 AND PROJECTED POPULATION DISTRIBUTION WITHIN THE LPZ INCLUDING TRANSIENT POPULATION

SECTOR DESIGNATION	1977 RADIAL INTERVAL (mi) 0-1	1-2	2-3
N	0	3	12,015 (15 + 12,000*)
NNE	3	3	34
NE	0	21	37
ENE	0	15	31
E	0	12	430 (15 + 415*)
ESE	3	9	24
SE	0	18	12
SSE	6	24	6
S	3	6	15
SSW	21	15	6
SW	9	12	21
WSW	3	18	46
W	0	6	37
WNW	3	3	6
NW	0	205	311 (266 + 45*)
NNW	0	9	9
Sum for Radial Interval	51	379	13,040 (580 + 12,460*)
Cumulative Total to Outer Radius	51	430	13,470 (1,010 + 12,460*)
Average Density (people/mi^2) in Radial Region	16	34	476 (36 + 440*)

*Denotes peak daily transient population only.

TABLE 2.1-5 (Cont'd)

SECTOR DESIGNATION	1980 RADIAL INTERVAL (mi) 0-1	1-2	2-3
N	0	3	12,016 (16 + 12,000*)
NNE	3	3	37
NE	0	23	40
ENE	0	16	34
E	0	13	431 (16 + 415*)
ESE	3	10	26
SE	0	20	13
SSE	6	26	7
S	3	6	17
SSW	22	16	6
SW	10	13	22
WSW	3	19	49
W	0	6	39
WNW	3	3	6
NW	0	217	326 (281 + 45*)
NNW	0	10	10
Sum for Radial Interval	53	404	13,079 (619 + 12,460*)
Cumulative Total to Outer Radius	53	457	13,536 (1,076 + 12,460*)
Average Density (people/mi^2) in Radial Region	17	36	478 (38 + 440*)

*Denotes peak daily transient population only.

TABLE 2.1-5 (Cont'd)

SECTOR DESIGNATION	1990 RADIAL INTERVAL (mi) 0-1	1-2	2-3
N	0	4	12,018 (18 + 12,000*)
NNE	4	4	45
NE	0	28	49
ENE	0	20	41
E	0	16	435 (20 + 415*)
ESE	4	12	32
SE	0	24	16
SSE	8	31	8
S	4	7	19
SSW	26	18	7
SW	11	15	26
WSW	4	22	56
W	0	7	45
WNW	4	4	7
NW	0	250	369 (324 + 45*)
NNW	0	11	11
Sum for Radial Interval	65	473	13,184 (724 + 12,460*)
Cumulative Total to Outer Radius	65	538	13,722 (1,262 + 12,460*)
Average Density (people/mi^2) in Radial Region	21	43	485 (45 + 440*)

*Denotes peak daily transient population only.

TABLE 2.1-5 (Cont'd)

SECTOR DESIGNATION	2000 RADIAL INTERVAL (mi) 0-1	1-2	2-3
N	0	4	12,020 (20 + 12,000*)
NNE	4	4	51
NE	0	31	55
ENE	0	22	46
E	0	18	437 (22 + 415*)
ESE	4	13	36
SE	0	27	18
SSE	8	34	9
S	4	8	21
SSW	29	20	8
SW	12	16	29
WSW	4	24	63
W	0	8	50
WNW	4	4	8
NW	0	279	407 (362 + 45*)
NNW	0	12	12
Sum for Radial Interval	69	524	13,270 (810 + 12,460*)
Cumulative Total to Outer Radius	69	593	13,863 (1,403 + 12,460*)
Average Density (people/mi^2) in Radial Region	22	47	490 (50 + 440*)

*Denotes peak daily transient population only.

TABLE 2.1-5 (Cont'd)

SECTOR DESIGNATION	2010 RADIAL INTERVAL (mi)		
	0-1	1-2	2-3
N	0	5	12,022 (22 + 12,000*)
NNE	5	5	57
NE	0	35	62
ENE	0	25	52
E	0	20	440 (25 + 415*)
ESE	5	15	40
SE	0	30	15
SSE	10	39	10
S	5	9	24
SSW	32	23	9
SW	14	18	32
WSW	5	27	69
W	0	9	56
WNW	5	5	9
NW	0	309	446 (401 + 45*)
NNW	0	14	14
Sum for Radial Interval	81	588	13,357 (897 + 12,460*)
Cumulative Total to Outer Radius	81	669	14,026 (1,566 + 12,460*)
Average Density (people/mi^2) in Radial Region	26	53	495 (55 + 440*)

*Denotes peak daily transient population only.

TABLE 2.1-5 (Cont'd)

SECTOR DESIGNATION	2020 RADIAL INTERVAL (mi)		
	0-1	1-2	2-3
N	0	5	12,024 (24 + 12,000*)
NNE	6	6	63
NE	0	39	69
ENE	0	28	58
E	0	22	443 (28 + 415*)
ESE	6	17	45
SE	0	33	22
SSE	11	43	11
S	5	10	26
SSW	35	25	10
SW	15	20	35
WSW	5	30	77
W	0	10	62
WNW	5	5	10
NW	0	342	489 (444 + 45*)
NNW	0	15	15
Sum for Radial Interval	88	650	13,459 (999 + 12,460*)
Cumulative Total to Outer Radius	88	738	14,197 (1,737 + 12,460*)
Average Density (people/mi^2) in Radial Region	28	59	501 (61 + 440*)

*Denotes peak daily transient population only.

TABLE 2.1-6

EDUCATION INSTITUTIONS WITHIN 10 MILES OF THE BYRON STATION

INSTITUTIONS	DISTANCE AND DIRECTION FROM SITE	GRADES	ENROLLMENT 1980-1981	STAFF
Byron, Illinois				
Byron Middle and High School	3.7 miles NNE	7-12	485	62
Mary Morgan Elementary School	3.7 miles NNE	K-6	504	46
Oregon, Illinois				
Public:				
Oregon High School	5.1 miles SSW	9-12	450	35
Etnyre Middle School	5.1 miles SSW	6-8	420	25
Jefferson Elementary School	5.1 miles SSW	3-5	278	19
Nash Elementary School	5.1 miles SSW	K-2	189	11
Oregon Annex - Special Education Building	5.1 miles SSW		65	9
Private:				
Village of Progress Inc.	5.1 miles SSW		92	18
Stillman Valley, Illinois				
Stillman Valley High School	5.7 miles ENE	9-12	487	30
Hale Jr. High School	5.7 miles ENE	6-8	360	23
Highland Elementary School	5.7 miles ENE	K-5	417	20

TABLE 2.1-6 (Cont'd)

INSTITUTIONS	DISTANCE AND DIRECTION FROM SITE	GRADES	ENROLLMENT 1980-1981	STAFF
Leaf River, Illinois				
Leaf River Community School	6.5 miles WNW	K-12	478	40
Mount Morris, Illinois				
Mount Morris High School	7.5 miles WSW	9-12	239	27
Mount Morris Jr. High School	7.5 miles WSW	6-8	179	15
Mount Morris Elementary School	7.5 miles WSW	K-5	385	29
Mount Morris Special Education Building	7.5 miles WSW		38	17

Sources: Apper (1981), Blakely (1981), Brown (1981), Glasser (1981), Lamb (1981), Maloney (1981), Miller (1981), Turner (1981).

TABLE 2.1-7

INDUSTRIES WITHIN 10 MILES OF THE BYRON STATION

NAME OF FIRM	LOCATION	EMPLOYMENT	PRODUCTS
Barker Lumber Company	Byron	2	lumber, hardware, building materials
Farmco Inc.	Byron	7	stone aggregates
Kysor Industrial Corporation	Byron	200	automotive accessories
Quality Metal Furnishing Company	Byron	185	metal finishing
Acme Resin Company	Oregon	60	foundry sand
Atwood Vacuum Machine Company	Oregon	75	automotive parking brakes
Carnation Milk Company	Oregon	22	liquid diet product
Caron Spinning Company	Oregon	42	knitting yarns
Cook Manufacturing Company	Oregon	40	machine parts
Dye Fixture & Display Co.	Oregon	10	store fixtures, display units
E.D. Etnyre & Company	Oregon	270	road building machinery
Martin Marietta Aggregates	Oregon	53	sand
Offset Preparation Service Inc.	Oregon	40	film for off-set printing
Oregon Ready-Mix	Oregon	7	concrete
Oregon Stone Quarries	Oregon	9	stone aggregates, asphalt
Paragon Foundries Company	Oregon	19	iron and bronze castings

TABLE 2.1-7 (Cont'd)

NAME OF FIRM	LOCATION	EMPLOYMENT	PRODUCTS
Republic Reporter Corporation	Oregon	7	printing
Rock Wood Carvers	Oregon	8	wood furniture
Sinnissippi Forest Sawmill	Oregon	6	lumber, wood products, fence posts, X-mas trees and greens
Wood Brothers, Inc.	Oregon	475	rotary mowers, rear blades, and discs
Woodhaven Industries	Oregon	1	front ends of racing cars
Stillman Valley Tool	Stillman Valley	11	gear manufacturing
Kable Printing Company	Mount Morris	500	printing
Kable News	Mount Morris	272	magazine distributor
Rowland Printing Company	Mount Morris	3	printing
Rude's Custom Butchering	Mount Morris	13	meat
Snyder Manufacturing Company	Mount Morris	30	electric equipment
Sterling Quality Products	Mount Morris	2	candies and syrups

Source: Telephone survey of individual industries by Marketing Department, Dixon District, Rock River Division, Commonwealth Edison Company, June 1981.

TABLE 2.1-8

POPULATION CENTERS WITHIN 50 MILES OF THE BYRON STATION

POPULATION CENTER	COUNTY	DISTANCE AND DIRECTION FROM SITE	1980* POPULATION
Rockford	Winnebago (Ill.)	16.8 miles NE	139,712
Freeport	Stephenson (Ill.)	22.3 miles NW	26,406
De Kalb	De Kalb (Ill.)	28.6 miles ESE	33,099
Beloit	Rock (Wis.)	32.1 miles NNE	35,207
Janesville	Rock (Wis.)	43.1 miles NNE	51,071
Clinton	Clinton (Iowa)	47.6 miles WSW	32,828

*Source: Bureau of the Census (1981).

TABLE 2.1-9

URBAN CENTERS WITHIN 30 MILES OF THE BYRON STATION

URBAN CENTER	COUNTY	DISTANCE AND DIRECTION FROM SITE	1980* POPULATION
Oregon	Ogle	5.1 miles SSW	3,559
Mount Morris	Ogle	7.5 miles WSW	2,989
Rochelle	Ogle	15.3 miles SE	8,982
Polo	Ogle	15.8 miles WSW	2,643
Rockford	Winnebago	16.8 miles NE	139,712
Dixon	Lee	18.6 miles SSW	15,659
Loves Park	Winnebago	20.2 miles NE	13,192
Freeport	Stephenson	22.3 miles NW	26,406
Belvidere	Boone	25.3 miles ENE	15,176
Sterling	Whiteside	27.7 miles SW	16,273
De Kalb	De Kalb	28.6 miles ESE	33,099
Rock Falls	Whiteside	28.9 miles SW	10,624
Genoa	De Kalb	29.9 miles E	3,276
Byron	Ogle	3.7 miles NNE	2,035

*Source: Bureau of the Census (1981).

TABLE 2.1-10

LIVESTOCK STATISTICS AND PRODUCTION

	OGLE COUNTY		WINNEBAGO COUNTY	
	1978	1979	1978	1979
All Cattle (no. of head)	90,100[a]	85,100[b]	44,300[a]	41,900[b]
Beef Cows (no. of head)	15,300[a]	15,200[b]	5,800[a]	5,400[b]
Milk Cows (no. of head)	7,600	7,900	6,100	6,100
Hogs & Pigs (no. of head)	155,800	164,800	69,800	76,300
Sheep (no. of head)	5,400[a]	6,200[b]	2,700[a]	3,100[b]
Poultry (no. of layers)	81,600	76,000	12,600	10,300
Egg Production (no. of eggs)	19,800,000	17,900,000	3,000,000	2,400,000
Milk Production (pounds)	78,300,000	79,900,000	62,800,000	61,700,000

Source: Illinois Cooperative Crop Reporting Service et al. (1980).

[a]As of January 1, 1979.

[b]As of January 1, 1980.

TABLE 2.1-11

NEAREST COW WITHIN A 5-MILE RADIUS OF THE BYRON STATION

DIRECTION	APPROXIMATE DISTANCE TO NEAREST COW (miles)
N	*
NNE	*
NE	1.9
ENE	1.5
E	2.4
ESE	3.1
SE	3.0
SSE	4.0
S	2.3
SSW	2.0
SW	*
WSW	2.0
W	2.5
WNW	3.1
NW	1.4
NNW	*

Note: Area surveyed on July 30 and August 3, 1981, by P. Coulter of Hazleton Environmental Sciences.

*None within 5 miles in this direction.

TABLE 2.1-12

NEAREST RESIDENCE AND GARDEN WITHIN A 5-MILE RADIUS OF THE BYRON STATION

DIRECTION	APPROXIMATE DISTANCE TO THE NEAREST RESIDENCE (miles)	APPROXIMATE DISTANCE TO THE NEAREST GARDEN* (miles)
N	1.1	1.1
NNE	1.5	1.5
NE	1.0	1.0
ENE	1.3	1.3
E	1.1	1.2
ESE	1.4	1.4
SE	1.4	1.4
SSE	0.9	0.9
S	0.8	0.8
SSW	0.7	0.9
SW	0.9	0.9
WSW	1.7	1.9
W	1.9	1.9
WNW	0.7	0.7
NW	1.0	1.3
NNW	1.4	1.5

Note: Area surveyed on July 30 and August 3, 1981, by P. Coulter of Hazleton Environmental Sciences.

*Of 500 ft^2

2.2 NEARBY INDUSTRIAL, TRANSPORTATION, AND MILITARY FACILITIES

2.2.1 Locations and Routes

2.2.1.1 Industrial Plants

Most of the industries within 10 miles of the plant are situated in Oregon, which is located 5.1 miles south-southwest of the site (centerline of the reactors). There are a few industries located in the cities of Byron and Mount Morris, and one in Stillman Valley. Facilities located in these cities are shown in Figure 2.2-1.

2.2.1.2 Military Bases and Missile Sites

There are no military bases within 10 miles of the site, although military landing rights are available in the cities of Rochelle, Rockford, and Freeport, all of which are farther than 10 miles from the site. Annual military operations for these airports have been estimated at 0 for the Rochelle Municipal Airport, 5430 for the Greater Rockford Airport, and 24 for the Albertus Airport in Freeport (Reference 1).

There are no missile bases within 50 miles of the site (Reference 2).

2.2.1.3 Pipelines

There are a number of gas and oil pipelines located within 10 miles of the site, as indicated in Table 2.2-1. Those located within 5 miles are shown in Figure 2.2-2. The closest pipeline to the site is located approximately 2.5 miles north of the site. It is a natural gas line, 3 inches in diameter, owned by Northern Illinois Gas Company.

2.2.1.4 Waterways

The Rock River is the major waterway for the area surrounding the Byron site, although it is considered nonnavigable to commercial traffic in this vicinity. The plant site is located 2.2 miles east of the Rock River.

2.2.1.5 Airports

There are four airports and one seaplane base within 10 miles of the site, as listed in Table 2.2-2 and shown in Figure 2.2-3. They are all private facilities and are used primarily for general aviation, as opposed to commercial flying. The closest airport to the site is located 1.5 miles to the south-southwest.

2.2.1.6 Transportation Routes

There are three low-altitude federal airways in the vicinity of the site, identified as V227, V127, and V172, which are used by aircraft operating below 18,000 feet mean sea level (MSL). Minimum altitudes for aircraft operating under instrument flight rules for the three airways are listed in Table 2.2-3. The centerlines for the three airways are located approximately 6 miles west (V227), 6 miles east (V127), and 8 miles south (V172) of the site, respectively. Figure 2.2-3 shows the site in relation to the three airways. Peak daily air traffic on each of the airways for 1976 is as follows: 12(V227), 26(V127), and 31(V172).

The highway transportation network and the traffic volumes of the area within 6 miles of the Byron site are shown in Figure 2.1-4. Relatively high traffic flow occurs on Illinois State Routes 2, 64, and 72, with 24-hour annual averages of 2000 cars and over in some sections. State Route 2 is especially well-traveled with 24-hour annual averages ranging from 4000 cars between Byron and Oregon to 8800 cars in Oregon.

The railway network within 6 miles of the Byron site is shown in Figure 2.1-5. There are three rail lines within this radius. The closest railroad, the Chicago, Milwaukee, St. Paul and Pacific, is located 4.0 miles north of the site. The Chicago and Northwestern Railroad and the Burlington Northern Railroad are located 4.5 miles north and 5.7 miles south of the site, respectively.

2.2.2 Descriptions

2.2.2.1 Description of Facilities

Table 2.1-7 lists all industries within 10 miles of the Byron station, their products, and their employment. The majority of these industries produce building materials, machinery, and machine parts for use in farming and construction. They vary in size from 1 employee (Woodhaven Industries) to 500 employees at the Kable Printing Company, which is the largest employer within the 10-mile radius. Most of the industries within 10 miles employ less than 25 people.

There are numerous mining and quarrying operations within 10 miles of the site, as indicated in Figure 2.2-1. The majority are small operations, but there are three quarries of substantial size near the city of Oregon. The quarries in this area produce sand, stone aggregates, and some asphalt and concrete.

2.2.2.2 Description of Products and Materials

Table 2.1-7 lists the products manufactured by the industries within 10 miles of the Byron Station. For the most part the materials used to produce these items are not hazardous or toxic.

Those companies which do use or store potentially hazardous materials are listed in Table 2.2-4, along with information concerning location, quantities used and/or stored, mode of transportation, and frequency of shipment.

Blasting occurs at some of the numerous quarries in the vicinity, but explosive materials are not stored onsite. Quarry companies subcontract all drilling and blasting to outside firms (References 3 and 4).

2.2.2.3 Pipelines

There are 15 pipelines within 10 miles of the site, of which 3 carry petroleum products and the remainder carry natural gas. The pipelines are listed in Table 2.2-1, along with pertinent information concerning location, size, age, burial depth, operating pressure, and valve types. There is no indication of any future changes in the type of product presently carried in these pipelines.

There are no tank farms within 10 miles of the site. The closest storage facility is located in Pecatonica, 16.3 miles north-northwest of the site, and is used by the Northern Illinois Gas Company for gas storage. The American Oil Company pipeline, which is located approximately 3 miles southwest of the site, has a facility identified as the Oregon Station. This facility is a pipeline booster station and is used to increase the capacity of the pipeline, not to store oil.

2.2.2.4 Waterways

Although the Rock River is considered nonnavigable to commercial traffic in the vicinity of the site, it is used heavily for recreational pursuits such as boating, fishing, and waterskiing. It is estimated that approximately 16,900 people use the river during March through November (Reference 5).

The closest dam to the site is located approximately 4.5 miles downstream in the city of Oregon.

2.2.2.5 Airports

The closest airport to the site is the Yost International Airport, located 1.5 miles south-southwest of the site. It is a private airport with one turf runway, 1700 feet in length and oriented from the northeast to the southwest (Reference 6). Although the length of the airstrip will be shortened due to the presence of transmission lines from the Byron Station, the airport will remain active. The Illinois Division of Aeronautics has determined that the airport will meet minimum requirements and the FAA designation of Yost Airport will not be changed. There are approximately 730 operations annually, all of which are private single engine planes (Reference 7).

Table 2.2-2 lists the airports within 10 miles of the site. There are four airports and one seaplane base, all of which are small private facilities. Figure 2.2-3 indicates the location of these airports relative to the site. None of these airports have ever had an aircraft accident to the knowledge of their owners, as indicated in Table 2.2-2. The landing and holding patterns associated with smaller airports are practically nonexistent, as most conduct operations with only one airplane at a time.

The nearest public airport is the Greater Rockford Airport, located 14 miles northeast of the site. There are an estimated 88,225 local operations and 93,610 itinerant operations annually, with a peak month total of 19,891 operations (Reference 8). Only one commercial airline, Ozark Airlines, flies into the Greater Rockford Airport.

2.2.2.6 Projections of Industrial Growth

There are no known plans for future industrial expansion in terms of facilities and inventories, but a few industries do expect an increase in consumption of their products. Depending on the amount of increased consumption, a change of plans to include expansion could occur but the most likely result would be an increase in production.

There are no known plans for expansion of the pipeline network within 10 miles of the site. There is also no indication that any of the airports within this radius will be expanding, except that those without based aircraft at the present time may acquire planes sometime in the future.

2.2.3 Evaluation of Potential Accidents

On the basis of the information provided in Subsections 2.2.1 and 2.2.2, safety evaluations of the activities described therein are provided in the following subsections.

2.2.3.1 Determination of Design Basis Events

The accident categories discussed below have been evaluated.

2.2.3.1.1 Explosions

No potential hazard involving the detonation of high explosives, munitions, chemicals, or liquid and gaseous fuels for facilities and activities in the vicinity of the plant where such materials are processed, stored, used, or transported in quantity has been found. The two roads in the vicinity used to transport explosives, River Road and German Church Road combined, never bear more than 15,000 to 20,000 pounds of dynamite or ammonium nitrate every 2 to 4 weeks. An evaluation for the effects of the explosion of shipments of this size is provided in Subsection 2.2.3.2.

2.2.3.1.2 Flammable Vapor Clouds (Delayed Ignition)

There is no possibility of an accident that could lead to the formation of flammable vapor clouds in the vicinity of the plant because (1) there is no industry in the vicinity of the plant which can produce a flammable vapor cloud, (2) there is no pipeline of sufficient size in the vicinity of the plant which can produce a flammable vapor cloud, and (3) there are no tank farms in the vicinity of the plant.

2.2.3.1.3 Toxic Chemicals

No significant potential for the release of toxic chemicals in the vicinity of the plant has been found. Those potentially hazardous materials used by local companies are listed in Table 2.2-4, with information concerning quantities stored, mode of transportation, and frequency of shipment. None of the potentially hazardous materials which are shipped on Route 2 are handled in sufficient quantity to have significant adverse effects on the plant as a result of accidental spillage or release at the plant site. There are no significant quantities of potentially hazardous chemicals stored on the plant site.

2.2.3.1.4 Fires

No fire hazard threatens the plant safety since no chemical plants, no large amounts of oil storage, and no gas pipelines are located in the vicinity of the plant. The potential for deleterious effects from forest or brush fires is minimized by the site's landscaping.

Onsite fire hazards are described in Subsection 9.5.1.

2.2.3.1.5 Collisions with Intake Structure

There is no potential for a barge or ship impact on the river screen house since the Rock River is nonnavigable in the vicinity of the site.

2.2.3.1.6 Liquid Spills

No potential for the accidental release of oil or liquids which may be corrosive, cryogenic, or coagulant, and which may be drawn into the plant's intake structure and circulating water system or which may otherwise affect the safety of the plant has been found.

2.2.3.2 Effects of Design Basis Events

As stated in Subsection 2.2.3.1.1, the maximum amount of dynamite transported on the two roads in the vicinity of the plant is 15,000 to 20,000 pounds. The design capability of the plant is such that a detonation of 60,000 pounds of TNT at the site boundary would cause no damage to safety-related plant structures. This design capability for the Byron Station comes about because of the fact that it is a duplicate of the Braidwood Station. The Braidwood Station has been designed to withstand the effects of an explosion of one boxcar load (132,000 pounds) of TNT at a distance of 1550 feet as discussed in Subsection 2.2.3.2 of the Braidwood UFSAR. For the smaller distance of 1200 feet at the Byron Station, the equivalent weight of TNT to produce the same effect is 60,000 pounds.

2.2.4 References

1. "Airport Master Records (FAA Form 5010-1)," Federal Aviation Administration, Department of Transportation: Greater Rockford Airport, March 4, 1977; Albertus Airport, December 15, 1976; and Rochelle Municipal Airport, October 28, 1976.

2. Major Jim Morrison, U.S. Department of the Army, phone conversation with C. W. Comerford, S&L Cultural Resource Analyst, May 18, 1977.

3. Thomas Klein, Secretary - Treasurer of Oregon Stone Quarries, Inc., letter to C. W. Comerford, S&L Cultural Resource Analyst, May 1977.

4. Jesse Copeland, Plant Manager of Martin Marietta Aggregates, Industrial Sand Division, letter to C. W. Comerford, S&L Cultural Resource Analyst, April 27, 1977.

5. Roy Hayes, Site Superintendent, Lowden State Park, phone conversation with C. W. Comerford, S&L Cultural Resource Analyst, June 14, 1977.

6. John Yost, Owner of Yost International Airport, phone conversation with C. W. Comerford, S&L Cultural Resource Analyst, April 21, 1977.

7. Michael C. Rose, Airports Planning Specialist, Federal Aviation Administration - Great Lakes Region, Department of Transportation, letter to C. W. Comerford, S&L Cultural Resource Analyst, April 15, 1977.

8. "Airport Master Records (FAA Form 5010-1)," Federal Aviation Administration, Department of Transportation, Greater Rockford Airport, March 4, 1977.

TABLE 2.2-1

OIL PIPELINES WITHIN 10 MILES OF THE SITE

COMPANY	NUMBER OF PIPELINES	DISTANCE AND DIRECTION FROM SITE	SIZE (in.)	AGE (years)	BURIAL DEPTH (ft)	MAXIMUM OPERATING PRESSURE	TYPE OF VALVES	APPROXIMATE SPACING	PRODUCTS
Amoco Pipeline Company*	1	3 miles SSW	10	30	2.5	1308 psi	Wescott 600# flanged OS & Y gate valves	1-17 miles apart	heating oils, diesel fuel, gasolines, jet fuel
Badger Pipeline Company**	1	5 miles E	8	30	3	400 psi	gate valves	--	propane
	1	5 miles E	12	14	3	1200 psi	gate valves	--	gasoline and fuel oils
Natural Gas Pipeline Company of America***	1	9 miles SE	20	36	3.5	800 psi	Nordstrom Plug Valves	15-20 miles apart	natural gas
	1	9 miles SE	24	12	3.5	800 psi	Cameron Ball	15-20 miles apart	natural gas

* Letter from Mr. R. L. Clapper, Chief Engineer, Amoco Pipeline Company, to C. W. Comerford, S&L Cultural Resource Analyst, April 27, 1977.

** Letter from Mr. H. N. Whitney, Superintendent of Operations, Badger Pipeline Company, to C. W. Comerford, S&L Cultural Resource Analyst, May 2, 1977.

*** Letter from Mr. J. E. Thompson, Vice President Engineering, Natural Gas Pipeline Company of America, to C. W. Comerford, S&L Cultural Resource Analyst, April 28, 1977.

TABLE 2.2-1 (Cont'd)

COMPANY	NUMBER OF PIPELINES	DISTANCE AND DIRECTION FROM SITE	SIZE (in.)	AGE (years)	BURIAL DEPTH (ft)	MAXIMUM OPERATING PRESSURE	TYPE OF VALVES	APPROXIMATE SPACING	PRODUCTS
Northern Illinois Gas Company†	1	5.2 miles NNE	22	18	2.5-3.5	600 psi	varied	varied	natural gas
	1	7 miles W	8	11-14	2.5-3.5	600 psi	varied	varied	natural gas
	2	6 miles SW	6	14-22	2.5-3.5	600 psi	varied	varied	natural gas
	3	varied	4	3-25	2.5-3.5	300-600 psi	varied	varied	natural gas
	3	varied	3	8-12	2.5-3.5	230-600 psi	varied	varied	natural gas

† Letter from Mr. J. Benavides, Northern Illinois Gas Company, to C. W. Comerford, S&L Cultural Resource Analyst, April 27, 1977.

TABLE 2.2-2

GENERAL INFORMATION FOR AIRPORTS WITHIN 10 MILES OF THE SITE*

AIRPORT	APPROXIMATE LOCATION FROM THE SITE	NUMBER OF BASED AIRCRAFT	HOURS	APPROXIMATE OPERATIONS**	RUNWAYS	SURFACE	LENGTH (ft)	WIDTH (ft)	AIRCRAFT ACCIDENTS
Yost International (Private)	1.5 miles SSW	1 single-engine	Unattended	two operations daily	13/31	Turf	1700	70	None
Lunn Seaplane Base (Private)	2.5 miles NW	0	Unattended	two operations daily during 9 months of the year	NE/SW	Water	--	--	None
Dana Blobaum Airport (Private)***	4 miles WNW	1 single-engine	Unattended	two operations per week	NE/SW	Turf	2100	80	None

* Source: Federal Aviation Administration, Airport Master Records (FAA Form 5010-1), Yost International, November 12, 1976, Lunn Seaplane Base, November 3, 1976, Duane E. Davis, November 10, 1976, and Stukenberg, December 4, 1976.

** Letter from Michael C. Rose, Airports Planning Specialist, Chicago Airports District Office, Federal Aviation Administration, to C. Comerford, S&L Cultural Resource Analyst, April 15, 1977.

*** Dana Blobaum, Airport Owner, Telephone Conversation with C. Comerford, S&L Cultural Resource Analyst, April 22, 1977.

TABLE 2.2-2 (Cont'd)

AIRPORT	APPROXIMATE LOCATION FROM THE SITE	NUMBER OF BASED AIRCRAFT	HOURS	APPROXIMATE OPERATIONS**	RUNWAYS	SURFACE	LENGTH (ft)	WIDTH (ft)	AIRCRAFT ACCIDENTS
Duane E. Davis Airport (Private)	5 miles WNW	1 single engine	Unattended	two operations daily	N/S	Turf	2200	150	None
Stukenberg Airport (Private)	7 miles WNW	0	Unattended	two operations daily	E/W	Turf	1800	150	None

TABLE 2.2-3

MINIMUM ALTITUDES FOR LOW-ALTITUDE FEDERAL AIRWAYS*

AIRWAY	SECTION	MINIMUM OBSTRUCTION CLEARANCE ALTITUDE (ft)	MINIMUM IN-ROUTE ALTITUDE (ft)
V 227	1	2500	2600
	2	2200	2700
	3	2200	2600
V 127	1	2700	2700
V 172	1	2400	2700
	2	2200	2700

* Source: Ms. Bonnie Ferguson, Operations Specialist, Federal Aviation Administration's General Aviation District Office, Telephone Conversation with J. M. Ruff, Sargent & Lundy Cultural Resource Analyst, April 19, 1977.

TABLE 2.2-4

INDUSTRIES WITH HAZARDOUS MATERIALS WITHIN 10 MILES OF THE SITE

INDUSTRY	LOCATION	MATERIAL	MAXIMUM QUANTITIES STORED	TOXICITY LIMIT†	TRANSPORTATION OF MATERIAL*
Farmco, Inc.	Byron	gasoline	NA	-	NA
Kysor Industrial Corp.	Byron	nitrogen	300 lb.	-	Rt. 2
		petroleum products	2 drums	-	Rt. 2
Quality Metal Furnishing Co.	Byron	chlorine	2,400 lb.	15 ppm	Rt. 2
		nitrogen	3,360 lb.	-	Rt. 2
		sulfur dioxide	1,800 lb.	10 ppm	Rt. 2
		sulfuric acid	250 lb.	2 mg/m3	Rt. 2
		petroleum products	3 drums	-	obtained locally
Acme Resin Co.	Oregon	anhydrous ammonia	16,000 gal.	100 ppm	Rt. 2, 4 times per month
		formaldehyde	20,000 gal.	10 ppm	transported on the Burlington Northern RR, 3 times per month
Atwood Vacuum Machine Co.**	Oregon	Stoddard solvent	1 drum	1,000 ppm	Rt. 2, once per year
		Visconorust	1 drum	-	Rt. 2, once per year

TABLE 2.2-4 (Cont'd)

INDUSTRY	LOCATION	MATERIAL	MAXIMUM QUANTITIES STORED	TOXICITY LIMIT†	TRANSPORTATION OF MATERIAL*
Carnation Co.**	Oregon	ethyl ether	15 gal.	800 ppm	NA
		petroleum ether	15 gal.	50 ppm	NA
Cook Manufacturing Co.	Oregon	nitrogen	100 lb.	-	Rt. 2
		gasoline	300 gal.	-	obtained locally
		petroleum products	10 drums	-	Rt. 2
E. D. Etnyre & Co.	Oregon	carbon dioxide	2,000 lb.	10,000 ppm	Rt. 64
		helium	500 lb.	-	Rt. 64
		gasoline	1,000 gal.	-	Rt. 64
		petroleum products	2,000 gal.	-	Rt. 64
Helle Hardwood's, Inc.	Oregon	sulfur dioxide	1 gal.	10 ppm	Rt. 64
		gasoline	7,800 gal.	-	Rt. 64
		petroleum products	2,000 gal.	-	Rt. 64
Paragon Foundries Co.	Oregon	acetone	1 gal.	2,000 ppm	obtained locally
		sulfuric acid	1 gal.	2 mg/m^3	Rt. 64

TABLE 2.2-4 (Cont'd)

INDUSTRY	LOCATION	MATERIAL	MAXIMUM QUANTITIES STORED	TOXICITY LIMIT†	TRANSPORTATION OF MATERIAL*
Paragon Foundries Co.	Oregon	gasoline	20 gal.	-	obtained locally
		petroleum products	565 gal.	-	Rt. 2
Rock Wood Carvers, Inc.	Oregon	gasoline	10 gal.	-	Rt. 2
		petroleum products	1,500 gal.	-	Rt. 2, 5 times per yr.
Sinnissippi Forest Sawmill	Oregon	gasoline	1,000 gal.	-	Rt. 64, once per month
		petroleum products	300 gal.	-	Rt. 64, once per month
		pentochlorophenol	200 gal.	1 mg/m^3	Rt. 64, once annually
Wood Brothers, Inc.	Oregon	acetaldehyde	NA	400 ppm	Rt. 2, once per week
		carbon dioxide	12,000 lb.	10,000 ppm	Rt. 2, 2 times per month
		gasoline	10,000 gal.	-	Rt. 2, 6 times per yr.
		propane	20,000 gal.	2,000 ppm	Rt. 2, 8 times per yr.

TABLE 2.2-4 (Cont'd)

INDUSTRY	LOCATION	MATERIAL	MAXIMUM QUANTITIES STORED	TOXICITY LIMIT†	TRANSPORTATION OF MATERIAL*
Kable Printing Co.**	Mt. Morris	carbon dioxide	12,000 lb.	10,000 ppm	Rt. 2, once per year
		sulfur dioxide	2,000 lb.	10 ppm	Rt. 2 or 64, once per year
		sulfuric acid	220 gal.	2 mg/m^3	Rt. 2, 4 times per yr.
		L.P. gas	500 gal.	2,000 ppm	Rt. 2, 5 times per yr.
		Roto Solvent	16,000 gal.	-	Rt. 2 or 64, 7 times per yr.
		various non-flammable petroleum products	1,000 gal.	-	Rt. 2 or 64, 24 times per yr.
		kerosene	200 gal.	-	Rt. 2, 2 times per yr.
		Apco Thinner	1,000 gal.	-	Rt. 2 or 64, 3 times per yr.
		Tolusub	360 gal.	-	Rt. 2 or 64, once per year
		printing inks (containing roto solvent)	30,000 gal.	-	transported daily on Rt. 64

TABLE 2.2-4 (Cont'd)

INDUSTRY	LOCATION	MATERIAL	MAXIMUM QUANTITIES STORED	TOXICITY LIMIT†	TRANSPORTATION OF MATERIAL*
Rude's Custom Butchering	Mt. Morris	gasoline	500 gal.	-	Pines Road
Synder Manufacturing Co.**	Mt. Morris	gasoline	1,000 gal.	-	NA

† Adapted from Sax, "Dangerous Properties of Industrial Materials".
* Materials transported by truck unless otherwise noted.

Sources: 1) Industrial Survey performed by Commonwealth Edison, June, 1977

**2) Letters to C. W. Comerford, S&L Cultural Resource Analyst, April 27, 1977 to May 19, 1977

NA - not available

2.3 METEOROLOGY

Section 2.3 provides a meteorological description of the Byron Station site and its surrounding areas. This includes a description of general climate, meteorological conditions used for design and operating-basis consideration, summaries of normal and extreme values of meteorological parameters, a discussion of the potential influence of the plant and its facilities on local meteorology, a description of the onsite meteorological measurements program, and short-term and long-term diffusion estimates. Detailed summaries of meteorological parameters are presented using data from Argonne National Laboratory (1950-1964), from the first-order National Weather Service Stations at Rockford, Illinois (1951-1976), Peoria, Illinois (1948-1976), and Chicago (Midway Airport), Illinois (1943-1976), and the meteorological towers at the Byron Station (1974-1976) and Carroll County Station (1974-1976) sites.

Based on the information presented in this section, it is concluded that there are no unusual local conditions that should adversely affect the plant operation, the dispersion of the plant effluents, or the dissipation of the plant waste heat.

2.3.1 Regional Climatology

2.3.1.1 General Climate

The Byron Station site is located in north central Illinois, approximately 17 miles southwest of Rockford, Illinois. General climatological data for the region were obtained from the United States Environmental Science Services Administration (ESSA) Climate of Illinois report (Reference 1) and from the Local Climatological Data Annual Summaries for the National Oceanic and Atmospheric Administration (NOAA) first-order weather stations at Rockford (located 12 miles northeast of the Byron site, Reference 2), and Midway, Chicago (located 80 miles east-southeast of the Byron site, Reference 3). The 15-year Climatological Summary for Argonne National Laboratory (located 65 miles east-southeast of the Byron Site, Reference 4) is also consulted for specific statistics. There are only minor climatological variations over the entire area. The Rockford weather station is the closest first-order National Weather Service station to Byron, and for most of the climatic statistics, the most representative.

The climate of northern Illinois is typically continental, with cold winters, warm summers, and frequent short-period fluctuations in temperature, humidity, cloudiness, and wind direction. The great variability in northern Illinois climate is due to its location in a confluence zone particularly during the cooler months between different air masses (Reference 5). The specific air masses which affect northern Illinois include maritime tropical air which originates in the Gulf of Mexico, continental tropical air which originates in Mexico and the southern Rockies, Pacific air which originates in the eastern North Pacific Ocean, and continental polar and continental arctic air which originate in Canada. As these air masses migrate from their source regions they may undergo substantial modification in their characteristics. Monthly streamline analyses of resultant surface winds suggest that air reaching northern Illinois most frequently originates over the Gulf of Mexico from April through August, over the southeastern United States from September through November and over both the Pacific Ocean and the Gulf of Mexico from December through March (Reference 5).

The major factors controlling the frequency and variation of weather types in northern Illinois are distinctly different during two separate periods of the year.

During the fall, winter, and spring months, the frequency and variation of weather types is determined by the movement of synoptic-scale storm systems which commonly follow paths along a major confluence zone between air masses, which is usually oriented from southwest to northwest through the region. The confluence zone normally shifts in latitude during this period, ranging in position from the central states to the United States - Canadian border. The average frequency of passage of storm systems along this zone is about once every 4 to 8 days. The storm systems are most frequent during winter and spring months, causing a maximum of cloudiness during these seasons. Winter is characterized by alternating periods of steady precipitation (rain, freezing rain, sleet, or snow) and periods of clear, crisp, and cold weather. Springtime precipitation is primarily showery in nature. The frequent passage of storm systems, presence of high winds aloft, and frequent occurrence of unstable conditions caused by the close proximity of warm, moist air masses to cold and dry air masses result in this season's relatively high frequency of thunderstorms. These thunderstorms on occasion are the source for hail, damaging winds, and tornadoes. Although synoptic-scale storm systems also occur during the fall months, their frequency of occurrence is less than in winter or spring. Periods of pleasant, dry weather characterize this season, which ends rather abruptly with the returning storminess which usually begins in November.

In contrast, weather during the summer months is characterized by weaker storm systems which tend to pass to the north of Illinois. A major confluence zone is not present in the region, and the regions weather is characterized by much sunshine interspersed with thunderstorm situations. Showers and thunderstorms are usually of the air mass type, although occasional outbreaks of cold air bring precipitation and weather typical of that associated with the fronts and storm systems of the spring months.

When southeast and easterly winds are present in northern Illinois, they usually bring mild and wet weather. Southern winds are warm and showery, westerly winds are dry with moderate temperatures, and winds from the northwest and north are cool and dry.

The prevailing wind is west-northwesterly at Rockford and westerly at Midway. Although these are the most frequent wind directions, a relatively good distributional character exists over the other directions. The monthly average wind speed is lowest in midsummer at both stations, with the prevailing wind directions occurring from the south-southwest at Rockford and the southwest at Midway. The monthly average wind speed is highest in late winter and early spring months at both stations, with the prevailing wind directions occurring from the east-northeast and west-northwest at Rockford and from the west at Midway.

Table 2.3-1 presents a summary of climatological data from meteorological stations surrounding the Byron site. The annual average temperature in the Byron area as represented by Rockford data is 48.1°F, with extreme temperatures having ranged from a maximum of 103°F to a minimum of -22°F. Maximum temperatures equal or exceed 90°F about 13 times per year while minimum temperatures are less than or equal to 0°F about 16 times per year.

Humidity varies with wind direction, being lowest with west or northwest winds and higher with east or south winds. At the Rockford station, the relative humidity is highest during the late summer, with the August mean humidity ranging from 90% in early morning to 57% at noon. In winter the mean humidity ranges from 78% in early morning to 65% at noon (Reference 2). Heavy fog with visibilities less than 1/4 of a mile is rare, occurring on the average 23 times a year, most frequently during the winter months (Reference 2).

Annual precipitation in the Byron area averages about 37 inches; for the 40-year period (1937-1976) annual precipitation has ranged from 23.25 inches in 1976 to 56.48 inches in 1973 (Reference 2). While 34% of the average annual precipitation occurs in the summer months of June through August and 64% occurs in the 6 months from April through September, no month averages less than 4% of the annual total. Monthly precipitation totals have ranged from 11.81 inches to 0.01 inch. The maximum 24-hour precipitation recorded in the area was 5.66 inches. Snowfall commonly occurs from November through March with an average of about 33 inches of snow annually. The monthly maximum and 24-hour maximum snowfall recorded were 22.7 inches and 10.9 inches, respectively. Points in northern Illinois average about 6 days of sleet per year, with an average of 2.3 hours of sleet on a sleet day (Reference 6).

Because of prevailing westerly flow, the influence of Lake Michigan on northern Illinois weather is not great. When winter's northeasterly winds blow across the lake, cloudiness is often increased in the Rockford area, and the temperatures are slightly higher than those westward around the Mississippi River. Conversely, in summer, the cooling effect of Lake Michigan is occasionally felt as far westward as Rockford.

The terrain in northern Illinois is relatively flat, and differences in elevation have no significant influence on the general climate or regional dispersion characteristics. However, the low hills and river valleys that do exist exert a small effect on nocturnal wind drainage patterns and fog frequency.

2.3.1.2 Regional Meteorological Conditions for Design and Operating Bases

2.3.1.2.1 Thunderstorms, Hail, and Lightning

Thunderstorms occur an average of 42 days per year at Rockford (1951-1976) and 40 days per year at Midway, Chicago (1943-1976) (References 2 and 3). They occur most frequently during months of June and July; 8 days per month at Rockford, and 7 and 6 days per month for June and July, respectively, at Midway. Both stations average 5 or more thunderstorm days per month throughout the season from May through August, and 1 or less thunderstorm day per month from November through February. A thunderstorm day is recorded only if thunder is heard. The observation is independent of whether or not rain and/or lightning are observed concurrent with the thunder (Reference 7).

A severe thunderstorm is defined by the National Severe Storms Forecast Center (NSSFC) of the National Weather Service as a thunderstorm that possesses one or more of the following characteristics (Reference 8):

a. winds of 50 knots or more,

b. hail 3/4 inch or more in diameter, or

c. cumulonimbus clouds favorable to tornado formation.

Although the National Weather Service does not publish records of severe thunderstorms, the above referenced report of the NSSFC gives values for the total number of hail reports 3/4 inch or greater, winds of 50 knots or greater, and the number of tornadoes for the period 1955-1967 by 1-degree squares (latitude x longitude). The report shows that during this 13-year period the 1-degree square containing the Byron Station site had 14 hailstorms producing hail 3/4 inch in diameter or greater, 28 occurrences of winds of 50 knots or greater, and 27 tornadoes.

At least 1 day of hail is observed per year over approximately 90% of Illinois, with the average number of hail days at a point varying from 1 to 4 (Reference 9). Considerable variation has occurred in these figures; annual extremes at a point have varied from no hail in certain years to as many as 14 hail days in others. About 80% of the hail days occur from March through August with spring (March through May) being the primary period. In northern Illinois, 53% of all hail days occur in the spring (Reference 9). Total hailstorm life at a point averages about 7 minutes, with maximum storm life reported as not over 20 minutes for Illinois (Reference 6).

The frequency of lightning flashes per thunderstorm day over a specific area can be estimated by using a formula given by J. L. Marshall (Reference 10), taking into account the distance of the location from the equator:

$$N = (0.1 + 0.35 \sin \varnothing) \times (0.40 \pm 0.20) \qquad (2.3\text{-}1)$$

where:

N = number of flashes to earth per thunderstorm day per km^2, and

Ø = geographic latitude.

For the Byron site, which is located at approximately 42° north latitude, the frequency of lightning flashes (N) ranges from 0.07 to 0.20 flashes per thunderstorm day per km^2. The value 0.20 is used as a conservative estimate of lightning frequency in the calculations that follow.

Taking the representative average number of thunderstorm days per year in the site as 42, the frequency of lightning flashes per km^2 per year is 8.4 as calculated below:

$$\frac{0.20 \text{ flashes}}{\text{thunderstorm day x km}^2} \times \frac{42 \text{ thunderstorm days}}{\text{year}} = \frac{8.4 \text{ flashes}}{\text{km}^2 \text{ x year}} \qquad (2.3\text{-}2)$$

For the probability of a lightning strike to safety-related structures, Marshall (Reference 10) gives the total attractive area (in meters^2) for a structure of length L, width W, and height H as:

$$LW + 4H (L + W) + 4H^2 \qquad (2.3\text{-}2a)$$

The attractive area for a structure depends on the magnitude of the lightning current and its frequency of occurrence. The formula for the total attractive area as given here assumes a lightning strike current intensity of 2×10^4 amperes with a 50% frequency of occurrence.

For the Byron Station, the smallest rectangle enclosing the reactor containment buildings is approximately 132.3 meters in length and 45.7 meters in width (see Byron Drawings M-5 and M-14). The height of the containment building is approximately 60.7 meters. It has been assumed that the height of the entire rectangle is 60.7 meters. This issues a realistic estimate of a lightning strike on the containment structures. The attractive area of the rectangle surrounding the containment buildings is therefore approximately 0.095 km^2.

The reactor containment buildings of Byron Station have a probability of being struck which is equivalent to:

$$\frac{8.4 \text{ flashes}}{\text{km}^2 \text{ yr}} \times 0.095 \text{ km}^2 = 0.798 \frac{\text{flashes}}{\text{yr}} \qquad (2.3\text{-}2b)$$

Hence, a conservative estimate of the recurrence interval for a lightning strike on the reactor containment buildings is:

$$\frac{1}{0.798 \text{ flashes/yr}} = 1.25 \text{ years/flash} \qquad (2.3\text{-}2c)$$

The area of the Byron Station site is approximately 1000 acres, or about 4.0 km^2. Hence the expected frequency of lightning flashes at the site per year is 34 as calculated below:

$$\frac{8.4 \text{ flashes}}{\text{km x year}} \times 4.0 \text{ km}^2 = \frac{34 \text{ flashes}}{\text{year}} \qquad (2.3\text{-}3)$$

2.3.1.2.2 Tornadoes and Severe Winds

Illinois ranks eighth in the United States in average annual number of tornadoes (Reference 11). Tornadoes occur with the greatest frequency in Illinois during the months of March through June. For the period 1916-1969, the publication "Illinois Tornadoes" (Reference 11) lists 38 tornadoes which occurred in the 8 county area (Ogle, Carroll, Stephenson, Winnebago, Boone, DeKalb, Lee, and Whiteside) surrounding and including the Byron Station site. Figure 2.3-1 shows the county distribution of tornadoes for the entire state for the same period of record. For Ogle County, the total number of tornadoes was six.

Tornadoes can occur at any hour of the day but are more common during the afternoon and evening hours. About 50% of Illinois tornadoes travel from the southwest to northeast. Slightly over 80% exhibit directions of movement toward the northeast through east. Fewer than 2% move from a direction with some easterly component (Reference 11).

The likelihood of a given point being struck by a tornado in any given year can be calculated using a method developed by H. C. S. Thom (Reference 12). Thom presents a map of the continental United States showing the mean annual frequency of occurrence of tornadoes for each 1-degree square (latitude x longitude) for the period 1953-1962. For the 1-degree square containing the Byron Station site, (approximately 3470 mi^2 in area), Thom computed an annual average occurrence of 1.0 tornadoes. Assuming 2.82 mi^2 is the average area covered by a tornado (Reference 12), the mean probability of a tornado occurring at any point within the 1-degree square containing the Byron site in any given year is calculated to be .0008. This converts to a mean recurrence interval of 1230 years. Using the same annual frequency but an average area of tornado coverage of 3.5 mi^2 (from Wilson and Changnon, Reference 11), the mean probability of a tornado occurrence is .0010.

More recent data (Reference 8) containing tornado frequencies for the period 1955-1967 indicates an annual tornado frequency of 2.1 for the 1-degree square containing the Byron site. This frequency, with Wilson and Changnon's average path area of 3.5 mi^2, results in an estimated mean tornado probability of .0021, with a corresponding mean return period of about 470 years.

The results were presented in order to provide a reasonable estimate of tornado probability without addressing the accuracy of the estimate. Because of uncertainties in regard to tornado frequency and path area data, the annual tornado probability for the Byron site area should be expressed as being in the range of .0010 to .0020, with a tornado return period of about 500 to 1000 years. However, a conservatively high estimate can be taken to be .0021 or 470 years.

For the period 1970-1977, the NOAA publication "Storm Data" lists 17 tornadoes which have occurred in the 8 county area (Ogle, Carroll, Stephenson, Winnebago, Boone, DeKalb, Lee, and Whiteside) surrounding and including the Byron Station site. The majority of these tornadoes were short in length, narrow in width, and weak in intensity. Four tornadoes, however, were severe enough to cause damage losses in excess of $50,000 in each case.

The most destructive tornado recorded during the period 1970-1977 in the vicinity of the Byron Station occurred on April 6, 1972, near Polo in Ogle County. An estimated $200,000 in damage losses was left in the wake of the tornado. Approximately 20 rural farm buildings, a home, and a trailer were severely damaged and four electrical transmission towers were bent to the ground. Winds to 100 mph were observed at Polo Airport where a hangar and a cement block building were destroyed. The structural damage implies that the maximum wind speed of this tornado was approximately 150 mph.

The tornado path length extended 40 km with an average width 45 meters. A conservative estimate of the tornado path area is 1.8 km^2.

The following are the design-basis tornado parameters (Reference 13) that were used for the Byron Station:

a. rotational velocity = 290 mph,

b. maximum translational velocity = 70 mph,

c. radius of maximum rotational velocity = 150 feet,

d. pressure drop = 3.0 psi, and

e. rate of pressure drop = 2.0 psi/sec.

The design wind velocity used for Seismic Category I structures at the Byron Station site is 85 mph considering a 100-year recurrence interval. For Seismic Category II structures, the governing design wind velocity used is 75 mph with a recurrence interval of 50 years. The design wind velocities for the 50-year and 100-year recurrence intervals are obtained from Figures 1 and 2 of the American National standard Building Code Requirements for Minimum Design Loads in Buildings" (Reference 14). The vertical velocity distribution and gust factors employed for the wind velocities are from Reference 13 for exposure Type C (see Subsection 3.3.1.).

2.3.1.2.3 Heavy Snow and Severe Glaze Storms

Severe winter storms, those that produce snowfall in excess of 6 inches and often are accompanied by damaging glaze, are responsible for more damage in Illinois than any other form of severe weather, including hail, tornadoes, or lightning (Reference 15). These storms occur on an average of five times per year in the state. The state probability for one or more severe winter storms in a year is virtually 100% while the probability for three or more in a year is 87%. During the 61-year period-of-record 1900 to 1960 used in a severe winter storm analysis (Reference 15), a typical storm had an average point duration of 14.2 hours. Data on the average areal extent of severe winter storms in Illinois show that they deposit at least 1 inch of snow over 32,305 mi^2, with more than 6 inches covering 7500 mi^2. The northwestern area of Illinois (including the Byron Station site) had 144 occurrences of a 6-inch snowstorm during the years 1900-1960. About 60 of these storms deposited more than 6 inches of snow in the Ogle County area. These frequencies are the highest of any region in the state (Reference 15).

Sleet or freezing rain can occur during the colder months of the year where rain falls through a very shallow layer of cold air from an overlying warm layer. The rain then freezes in the air, causing sleet, or upon contact with the ground or other objects, causing glaze.

In Illinois during the 61-year period 1900-1960, there were 92 glaze storms defined either by the occurrence of glaze damage or by occurrence of glaze over at least 10% of Illinois, and these 92 represent 30% of the total winter storms (Reference 15). The greatest number of glaze storms in 1 year was 6 (1951); in 2 years, 9 (1950-1951); in 3 years, 10 (1950-1952); and in 5 years, 15 (1948-1952). In an analysis of these 92 glaze storms, Changnon (Reference 15) reports that in 66 storms, the heaviest glaze layers disappeared within 2 days; in 11 storms, 3 to 5 days; in 8 storms, 6 to 8 days; in 4 storms 9 to 11 days; and in 3 storms, 12 to 15 days. Fifteen days was the maximum persistence of glaze. Within the northern third of Illinois, eight localized areas received damaging glaze in an average 10-year period; Ogle County averages about 2 days of glaze per year (Reference 15).

Ice measurements recorded in some of the most severe Illinois glaze storms are shown in Table 2.3-2 (Reference 15). The listing reveals the occurrence of large radial thicknesses at various locations throughout Illinois and indicates that quite severe glazing can occur at any part of the state. An average of one storm every 3 years will produce glaze ice 0.75 inch or thicker on wires (Reference 15).

Strong winds during and after a glaze storm greatly increase the amount of damage to trees and power lines. In studying wind effects on glaze-loaded wires, the Association of American Railroads (Reference 16) concluded that maximum wind gusts were not as significant (harmful) a measure of wind damage potential as were speeds sustained over 5-minute periods. Moderate wind speeds occurring after glaze storms are most prevalent; however, speeds of 25 mph or higher are not unusual, and there have been 5-minute winds in excess of 40 mph with glaze thickness of 0.25 inch or more (Reference 15). Specific glaze thickness data for the five fastest 5-minute speeds, and the speeds with the five greatest measured glaze thickness measured in the after-storm periods of 148 glaze storms throughout the country during 1926-1937 are shown in Table 2.3-3. Although these data were collected from various locations throughout the United States, they are considered applicable design values for locations in Illinois.

The roofs of safety-related structures are designed to withstand the snow and ice loads due to winter probable maximum precipitation (PMP) with a 100-year recurrence interval antecedent snowpack. The 100-year return period snowpack weight of 28 psf (27 inches of snowpack) was obtained from the American National Standard building code requirements (Reference 14). The weight of the accumulation of the winter PMP from a single storm is 76 psf (14.6 inches of precipitable water, or about 146 inches of fresh snow), which was taken as the 48-hour PMP during the winter months (December through March) (Reference 17). The design-basis snow and ice load is then 104 psf (see Subsection 2.4.2).

2.3.1.2.4 Ultimate Heat Sink Design

The ultimate heat sink at Byron consists of two wet mechanical draft cooling towers and their associated makeup system. In order to evaluate the ultimate heat sink, 30 years of meteorological data is required. Long-term data most representative of the conditions at Byron Station were recorded at Rockford. However, the Rockford NWS station has only a 28-year period of record (1950-1977). Other than Rockford, data most representative of the meteorological conditions of the Byron site and not affected by large water bodies yet still providing a conservative evaluation of the ultimate heat sink were recorded at Peoria for a 30-year period (1948-1977). Peoria data extracted from National Oceanic and Atmospheric Administration (NOAA) 3-hourly observations on magnetic tape per Reference 18 were used in evaluating the heat dissipation characteristics of the proposed wet mechanical draft cooling towers under adverse atmospheric conditions. Peoria weather data was not available for January 1952 through December 1956. The decision was made to fill this data gap with meteorological data which best reflected the conditions at Peoria. Therefore, data from Springfield, the closest NWS station to Peoria, were used to complete the 30-year meteorological data record.

Average monthly temperature and humidity are summarized in Tables 2.3-43 and 2.3-44 for the representative meteorological data from Springfield and Peoria. Included for comparison are meteorological data from the Byron site and from Rockford.

The UHS tower is designed to fulfill its purpose under the extreme environmental conditions set forth in Regulatory Guide

1.27. The meteorological data from Peoria were employed to identify the period of meteorological record resulting in the minimum heat transfer to the atmosphere and maximum plant intake temperature. The Peoria weather tape was also used for a water consumption analysis to verify the availability of a 30-day cooling water supply.

The design UHS tower outlet temperature is 100°F. The design bases analysis performed for the UHS is described in Section 9.2.5.3.1.1. This analysis includes scenarios with the highest three-hour wet-bulb temperature, 82°F, which was recorded on July 30, 1961, at 3:00 pm. Per Regulatory Guide 1.27, the ultimate heat sink must be capable of performing its cooling function during the design basis event for this worst case three-hour wet-bulb temperature. However, the design operating wet-bulb temperature of the ultimate heat sink is 78°F (ASHRAE 1% exceedance value). The maximum heat rejection to the UHS is from the safe shutdown of two 3586.6-MWt (guaranteed core thermal power) PWR reactors, as a result of a loss-of-coolant accident (LOCA) concurrent with a loss of offsite power (LOOP) and the concurrent orderly shutdown and cooldown from maximum power to cold shutdown of the other unit using normal shutdown operating procedures. The accident scenario also includes a single failure. The maximum predicted UHS tower outlet temperature from the tower performance curves for this 3-hour analysis is 100°F.

The maximum water makeup rate required by the ultimate heat sink was determined using the maximum 1-day evaporation period (average dry bulb temperature = 90.5°F and average wet bulb temperature = 73.0°F) which was recorded on July 18, 1954. The maximum evaporative period was defined as the period having the maximum difference between dry bulb temperature and dew-point temperature.

Byron has more than a 30-day supply of water because it has a continuous makeup supply from the Rock River using the Seismic Category I makeup system. In the event that makeup from the Rock River is not available, an alternative makeup source is from the onsite deep wells. There are two deep wells which have been demonstrated to be able to supply water at a rate of 800 gpm per well for more than 30 days.

For details of the ultimate heat sink design and makeup water availability, see Subsections 9.2.5 and 2.4.11.6.

2.3.1.2.5 Inversions and High Air Pollution Potential

Thirteen years of data (1952-1964) on vertical temperature gradient from Argonne (Reference 4) provide a measure of thermodynamic stability (mixing potential). Weather records from many U.S. stations have also been analyzed with the objective of characterizing atmospheric dispersion potential (References 19 and 20).

The seasonal frequencies of inversions based below 500 feet for the Byron Station are shown by Hosler (Reference 19) as:

Season	% of Total Hours	% of 24-Hour Periods With at Least 1 Hour of Inversion
Spring	30	71
Summer	31	81
Fall	37	68
Winter	31	53

Since northern Illinois has a primarily continental climate, inversion frequencies are closely related to the diurnal cycle. The less frequent occurrence of storms in summer produces a larger frequency of nights with short-duration inversion conditions.

Holzworth's data (Reference 20) give estimates of the average depth of vigorous vertical mixing, which give an indication of the vertical depth of atmosphere available for mixing and dispersion of effluents. For the Byron Station region, the seasonal values of the mean daily mixing depths (in meters) are:

	Mean Daily Mixing Depths	
Season	Morning	Afternoon
Spring	480	1400
Summer	300	1600
Fall	390	1200
Winter	470	580

When daytime (maximum) mixing depths are shallow, pollution potential is highest.

Argonne data are presented below in terms of the frequency of inversion conditions in the 5.5- to 144-foot layer above the ground as percent of total observations and in terms of the average duration of inversion conditions.

Month	Inversion Frequency	First Hour	Final Hour
January	30.5%	5 p.m.	8 a.m.
April	33.1%	6 p.m.	6 a.m.
July	42.4%	6 p.m.	6 a.m.
October	48.4%	5 p.m.	7 a.m.

Nocturnal inversions begin at dusk and normally continue until daylight the next day. The inversion frequency for January at Argonne compares well with Hosler's winter value and the fall season shows a maximum in both Argonne and Hosler's data. Fall also has the longest period of inversion conditions.

Holzworth has also presented statistics on the frequency of episodes of high air pollution potential, as indicated by low mixing depth and light winds (Reference 20). His data indicate that, during the 5-year period 1960-1964, the region including the Byron Station site experienced no episodes of 2 days or longer with mixing depths less than 500 meters and winds less than 2 m/sec. There were two such episodes with winds remaining less than 4 m/sec. For mixing heights less than 1000 meters and winds less than 4 m/sec there were about nine episodes in the 5-year period lasting 2 days or more but no episodes lasting 5 days or more. Holzworth's data indicate that northern Illinois is in a relatively favorable dispersion regime with respect to low frequency of extended periods of high air pollution potential.

2.3.2 Local Meteorology

2.3.2.1 Normal and Extreme Values of Meteorological Parameters

Meteorological data from the Greater Rockford Airport (12 miles northeast of the Byron site) and the Argonne National Laboratory (65 miles east-southeast of the Byron site) are used as regional data for the Byron Station site. Rockford data have been extracted from NOAA Local Climatological Data Summaries and 3-hourly observations on magnetic tape per Reference 18 for the 10-year period 1966 to 1975. A 15-year climatological summary compiled by Argonne National Laboratory from 1950 to 1964 (Reference 4) is used as a comparative long-term data base. Onsite meteorological data available for the period-of-record January 1, 1974 through December 31, 1976 are summarized and presented in this section wherever possible. Some meteorological data from the onsite meteorological program at the Carroll County Station site (37 miles west of the Byron Station site) from January 1, 1974 to December 31, 1976 are also used as a short-term data base for comparison with the Byron data.

2.3.2.1.1 Winds

Detailed wind records from the onsite meteorological measurements program for the period-of-record January 1974 through December 1976 were used to prepare wind roses. The monthly and annual period of record wind roses for the 30-foot level are presented in Figures 2.3-2 through 2.3-14. The annual period-of-record wind rose for the 250-foot tower level is presented in Figure 2.3-15. The period-of-record wind roses and persistence of wind direction data for both tower levels are in tabular form in Tables 2.3-4 through 2.3-6.

Seasonal wind roses at the 19-foot level for Argonne (1950-1964) are given in Figures 2.3-16 through 2.3-19. The annual surface wind rose for Rockford (1966-1975) is given in Figure 2.3-20. Wind direction persistence data at Argonne and Rockford for the same period and levels are presented in Table 2.3-7 and Table 2.3-8, respectively.

The predominant wind directions at the Byron site are south clockwise to southwest, occurring in these sectors more than 27% of the time at both tower levels for the period-of-record January 1974 through December 1976. Seasonal variations are evident from monthly data. The 30-foot level wind roses (Figures 2.3-2 through 2.3-14) indicate that January and February have moderately strong winds from the sector extending from the south clockwise to the northwest. March and April experience the greatest frequency of winds with speeds higher than 7.0 meters per second, which occur primarily from the south, south-southwest, northwest, and east-northeast directions. In May, winds are moderate predominantly from the east-northeast and east. The rest of the year has a high frequency of winds with speeds less than 1.5 meters per second prevailing from the south clockwise to southwest.

Calms at the 30-foot tower level occurred during 1.89% of the year, most frequently during July, August, and September; the longest persistence of calm conditions lasted for 11 hours during November. Table 2.3-5 shows that the persistence of one wind direction at the 30-foot level is usually less than 3 hours; the longest wind direction persistence occurred from the north during April for 42 hours.

The 250-foot level period-of-record wind rose at Byron (Figure 2.3-15) appears quite similar to the Byron 30-foot period-of record wind rose (Figure 2.3-14) in regard to the directional distribution of the winds, suggesting that the low level winds are not affected by nearby topography or vegetation. The upper level's prevailing wind was from the south-southwest 10.43% of the time, while the lower level's prevailing wind was from the south 9.93% of the time. The frequency of occurrence of winds greater than 7.0 m/sec in speed is substantially greater at the 250-foot level (48.3% of the time) than that experienced at the 30-foot level (14.5% of the time). Calms occur 0.18% of the time at the 250-foot level, less frequent and less persistent than at the 30-foot level.

For the long-term period of record 1950-1964, the Argonne station had a prevailing wind from the southwest but with substantial evident seasonal variations (Figures 2.3-16 though 2.3-19). There is a greater frequency of westerly to northwesterly winds in winter than in summer; the latter season has southwest and northeast winds which predominate. Argonne lies approximately 20 miles west-southwest of Lake Michigan while the Byron site is over 50 miles farther from the lake. An increase in occurrences of northeasterly winds at Argonne during the spring and summer as compared to the Byron wind statistics indicates that the Argonne station is influenced on occasion by lake effects from Lake Michigan. All winds with speeds less than 3 mph (1.3 m/sec) are classified as calm at Argonne; calms are least common in spring and most common in summer. The longest persistence of calm conditions observed at Argonne at the 19-foot level was 12 hours; at the 150-foot level, 9 hours.

The annual Rockford surface wind rose for the long-term period of record 1966-1975 (Figure 2.3-20) is similar to the lower level Byron Station wind rose, indicating the Byron data's representativeness of the surrounding area.

Rockford's prevailing wind is from the south and occurs with an annual frequency of 13.39%. The directional distribution of the wind for the remaining 15 sectors is fairly even with a frequency for each sector approximately between 3% and 8%. The frequency of calms at Rockford 20-foot level is 5.43%, which is greater than that at the Byron 30-foot level (1.89%). The increase of measured calm conditions at Rockford is due, in part, to Rockford's lower measuring height (20 feet) and less sensitive instrumentation (first-order NOAA weather station anemometers typically having starting speeds around 3 mph).

2.3.2.1.2 Temperatures

Monthly, annual average, and extreme temperature data at the Byron site for the short-term period of record January 1, 1974 through December 31, 1976 are compared in Table 2.3-9 with short-term periods of record from Rockford, Illinois (1973-1975) and the Carroll County Station site (1974-1976) in order to show the representativeness of the Byron site data for the region.

Table 2.3-10 presents, by month long-term average and extreme temperatures for Rockford (1966-1975) and Argonne (1950-1964), as compared to short-term average and extreme temperatures at the Byron Station site (1974-1976). These data are presented to indicate long-term temperature trends in the site region.

Temperature measurements at Rockford were made at the National Weather Service standard height of 4.5 feet above the surface. Temperatures from the 5.5-foot level at Argonne and the 33-foot level at the Carroll County site were used. In accordance with the requirement of NRC Regulatory Guide 1.23, temperature measurements at the Byron site were taken at a height of 30 feet.

Using a 3-year period of record for Rockford (1973-1975) and the Carroll County site (1974-1976), average temperatures of 48.7°F and 48.5°F were calculated, respectively. From 1974 through 1976, the 30-foot level at the Byron site averaged 47.3°F, indicating the representativeness of the Byron data for the general region.

The maximum temperature reported at the Rockford and Carroll County sites for their respective 3-year periods of record were 97°F and 93.4°F, respectively. The minimum temperatures reported at these stations were -21°F and -13.5°F, respectively. Maximum and minimum temperatures for the Byron Station site (1974-1976) were 93.2°F and -14°F. The temperature extremes measured at the Carroll County site and the Byron site are quite similar, suggesting these extremes are indicative of the temperature extremes experienced in the surrounding area. The larger range of temperature extremes reported at the Rockford site is attributed to a difference in measuring height, and a different period of record.

Long-term temperature averages for Rockford (1966-1975) and Argonne (1950-1964) were 47.8°F and 48°F, respectively. Temperatures at the Byron site (1974-1976) averaged near 47.3°F. Long-term extreme temperatures for Rockford were 98°F and -22°F, and for Argonne 101°F and -20°F. Extremes for the Byron site were 93.2°F and -14.7°F. The larger temperature ranges measured at the Rockford and Argonne stations are attributed, in part, to their lower measuring height and to their longer period of record.

Average daily maximum and minimum temperatures and average diurnal temperature ranges for each month at Rockford (1966-1975) are given in Table 2.3-11. Diurnal temperature variations average near 19°F for the year at Rockford; the cold months average a diurnal temperature range near 15°F while the warmer months experience diurnal temperature ranges closer to 21°F.

The design-basis maximum and minimum air temperatures considered in the design of systems and components are listed in Table 3.11-2. The various equipment components and associated environmental zones are listed in Table 3.11-1.

The extreme values of temperature for the long-term (1951-1976) at Rockford are given in Table 2.3-45. Also included are long-term average temperature values.

2.3.2.1.3 Atmospheric Moisture

Relative humidity data have been processed for the Byron site for the period 1974-1976. Extensive checking and editing of the data revealed that the overall recovery of relative humidity data the 30-foot and 250-foot levels were very low. Each of the calendar years 1974-1976 exhibited months where the data recovery rate was 0.00%. The combined relative humidity data recovery rate for August 1974, August 1975, and August 1976 was 0.00%. For all three August months, the relative humidity sensor was out for maintenance. In August 1974, the entire month of data was lost due to a sensor and pulse transmitter malfunction. In August 1975, no data were recorded because the relative humidity sensor had been sent to the manufacturer for a calibration check. In August 1976, the entire month of data was lost due to a lightning strike which rendered severe damage to the tower in mid-July 1976 and incapacitated the relative humidity sensor throughout the month of August 1976.

Therefore, a synthetic year of onsite data was formulated from those months of the 3-year period which exhibited the highest relative humidity recoveries: January through April 1974, May through July 1975, and September through December 1976.

To validate this rationale, the average monthly relative humidities from Rockford for the period 1974-1976 were compared to the average monthly relative humidities for the synthetic year of Byron data. Since the year to year variation of relative humidity at Rockford in most cases was not great, it was concluded that relative humidity data for any given month at Byron should adequately represent average relative humidity conditions for that month for the 3-year period 1974-1976. One year of data from Carroll County site (1976) is also presented to show the representativeness of the Byron site data for the region. Atmospheric moisture data from Rockford (1966-1975) and Argonne (1950-1964) are presented as an indication of long-term atmospheric moisture content trends for the region surrounding the Byron site.

Since the synthetic year of onsite relative humidity data is considered representative of the 3-year period 1974-1976, it can be concluded that the synthetic year represents the expected long-term conditions as well as the full 3-year 1974-1976 relative humidity data would have if the data were available. The intent of presenting short-term data (3 years) is not to represent long-term conditions. Rather, short-term data are intended to identify recent relative humidity trends in relation to the long-term record. A comparison of Tables 2.3-12 and 2.3-13 illustrates the short-term and long-term relative humidity conditions representative of the Byron site.

Table 2.3-48 summarizes the average monthly relative humidities for the 250-foot level at Byron. The data presented in Table 2.3-48 represent the moisture conditions relevant in the assessment of atmospheric impacts due to natural draft cooling tower operation.

2.3.2.1.3.1 Relative Humidity

The relative humidity for a given moisture content of the air is defined as the ratio of the actual mixing ratio of water vapor to that which would exist at saturation at the same temperature. The diurnal variation of relative humidity for a given moisture content of the air is inversely proportional to the diurnal temperature cycle. A maximum in relative humidity usually occurs during the early morning hours, while a minimum is typically observed in mid-afternoon.

Relative humidity data for the 30-foot level at Byron and the 33-foot level at Carroll County (1976) are presented in Table 2.3-12. The annual average relative humidities observed at Byron and Carroll County were 72.3% and 65.1%, respectively. The maximum relative humidity observed at Byron and Carroll County was 100% for most months throughout the year; the minimum relative humidity observed at both sites occurred during the month of October, with Byron having the lowest relative humidity value recorded of 6.5%.

Long-term relative humidity data for Rockford (1966-1975) and Argonne (1950-1964) are presented in Table 2.3-13. Argonne and Rockford relative humidity averages throughout the year were 74.9% and 72.0%, respectively. The maximum relative humidity observed at Rockford was 100% for all months, while the minimum relative humidity observed during the period of record was 15% observed in the months of March and April. These extremes are comparable with the short-term extremes observed at Byron and Carroll County. The annual average daily maximum and minimum relative humidities at Rockford were about 87% and 55%. The monthly average diurnal relative humidity range is greatest in May and least in December.

2.3.2.1.3.2 Dew-Point Temperature

The dew-point temperature is another measure of the amount of water vapor in the atmosphere. It is defined as the temperature to which air must be cooled to produce saturation with respect to water vapor, with pressure and water vapor content remaining constant.

Dew-point temperature data for the 30-foot Byron and 33-foot Carroll County levels are presented in Table 2.3-14. Monthly average dew-point temperatures at Byron ranged from 8.4°F in December to 62.5°F in July; Carroll County monthly averages ranged from 12.0°F in December to 59.1°F in July. The maximum dew-point temperatures at Byron and Carroll County were 87.0°F and 71.4°F respectively; the minimums were -20.0°F and -16.4°F, respectively.

Long-term dew-point temperature data for Rockford and Argonne are presented in Table 2.3-15. The annual dew-point temperature at Rockford (38.5°F) and Argonne (38.7°F) are generally comparable with the short-term averages at Byron (33.4°F) and Carroll County (35.4°F). Monthly average dew-point temperatures at Rockford ranged from 13.8°F in January to 60.6°F in July. The maximum dew-point temperature during the 10-year period was 79°F, while the minimum dew-point temperature was -31°F.

The annual average daily maximum and minimum dew-point temperatures at Rockford (1966-1975) were 44°F and 34°F. The maximum average diurnal variation in dew-point temperature was 15°F in January, while the minimum average diurnal variation was 7°F in July and August.

2.3.2.1.4 Precipitation

Precipitation data are available from the Byron site, Rockford, and Argonne and serve to indicate the precipitation characteristics of the Byron Station site.

2.3.2.1.4.1 Precipitation Measured as Water Equivalent

Monthly precipitation totals at Byron and Rockford for the short-term period-of-record January 1974 through December 31, 1976 are compared in Table 2.3-16. The precipitation totals at both stations for each month during the period-of-record shown are generally similar, indicating the representativeness of the Byron site data for the region. The heaviest monthly rainfall typically occurred during the spring; the maximum monthly rainfall for each location during the 3-year period was 7.39 inches at Byron and 6.98 inches at Rockford, both during May 1974. The lightest monthly rainfall generally occurred during the fall and winter months; the minimum monthly rainfall was 0.20 inches in November 1976 at Byron and 0.35 inches in September 1974 at Rockford.

Long-term normal and extreme monthly and yearly values of precipitation (water equivalent) in inches for Rockford (1966-1975) and Argonne (1950-1964) are shown in Table 2.3-17. The maximum and minimum monthly precipitation levels recorded for Rockford (1966-1975) are 10.63 inches during September 1973 and 0.04 inches during February 1969, respectively. The maximum and minimum monthly precipitation levels recorded for Argonne (1950-1964) are 13.17 inches during September 1954 and 0.03 inches during January 1961, respectively. In general, more than twice as much precipitation falls during the warmer summer months than during the colder winter months, due to increased convective storm activity during the warmer months.

The annual average precipitation was 39.85 inches for Rockford and 31.49 inches for Argonne. The differences in precipitation averages and extremes measured at the Rockford and Argonne Stations are attributed, in part, to the stations' differences in geographical location and nonoverlapping periods of record.

The number of hours of precipitation averaged per month at Rockford (1966-1975) and Argonne (1950-1964) are as follows:

Month	Rockford	Argonne
January	137	89
February	107	75
March	111	99
April	90	94
May	77	65
June	56	55
July	41	58
August	34	37
September	56	40
October	69	55
November	109	73
December	175	95
Yearly Average	1062	834

The previous summary shows that the larger summer precipitation amounts fall during fewer hours as compared to the relatively lesser winter precipitation levels. This reflects the fact that summer precipitation, associated with an increase in convective storm activity, is generally heavy and brief while winter precipitation is less intense and occurs over a longer period of time. These data indicate that Argonne's approximately 20% fewer hours of precipitation is consistent with Argonne's 20% less yearly average precipitation totals.

Table 2.3-18 presents the joint frequency distribution of wind direction and precipitation occurrence for Rockford (1966-1975). During the winter precipitation occurs most often with northerly winds, but there is good distribution of precipitation over all the other wind directions. Summer months have precipitation occurring most often with winds from the south. On an annual basis, precipitation occurs most often with winds from the north and south (1.2% of the time for each direction).

Maximum precipitation totals (water equivalent) in inches recorded for specified time intervals at Argonne (1950-1964) are shown in Table 2.3-19. The maximum 1-hour and 48-hour duration precipitation totals recorded were 2.20 inches in June 1953 and 8.62 inches in October 1954, respectively.

The extreme values of precipitation for the long-term (1951-1976) at Rockford are given in Table 2.3-46. Also included are long-term average precipitation values.

2.3.2.1.4.2 Precipitation Measured as Snow or Ice Pellets

The monthly average and the maximum monthly and 24-hour recorded values of snow and/or ice pellet precipitation for Rockford (1966-1975) are presented in Table 2.3-20. The maximum monthly value for the entire period is 20.0 inches occurring in December 1973. The maximum 24-hour value for the entire period is 7.6 inches occurring in February 1974.

The extreme values of ice and snow precipitation for the long-term (1951-76) at Rockford are given in Table 2.3-47. Also included are long-term average ice and snow precipitation values.

2.3.2.1.5 Fog

Fog is an aggregate of minute water droplets suspended in the atmosphere near the surface of the earth. According to international definition, fog reduces visibility to less than 0.62 mile (Reference 7). Fog types are generally coded as fog, ground fog, and ice fog in observation records. Observing procedures by the National Weather Service define ground fog as that which hides less than 0.6 of the sky and does not extend to the base of any clouds that may be above it (Reference 7). Ice fog is composed of suspended particles of ice. It usually occurs in high latitudes in calm clear weather at temperatures below -20°F and increases in frequency as temperature decreases (Reference 7).

Fog forms when the ambient dry bulb temperature and the dew-point temperature are nearly identical or equal. The processes by which these temperatures become the same and fog occurs are either by cooling the air to its dew point or by adding moisture to the air until the dew point reaches the ambient dry bulb temperature. This latter process is of particular interest with respect to cooling facility operation at power generating stations.

Cooling facility fog generally occurs when atmospheric conditions are conducive to natural fog formation. Natural processes such as radiational cooling at night or the advection of moist air are generally contributing factors. Thus the previous summary of natural fog occurrence is important to the understanding of the potential fogging problems for a proposed plant site.

Data on fog frequency and duration are presented for Rockford (1966-1975) in Table 2.3-21. Onsite data are not available to assess the fog characteristics at the Byron site. There are approximately 1097 hours of fog per year at Rockford. December has the greatest frequency of fog occurrence, with an average of approximately fifty-five 3-hourly observations recording fog. Fog occurs least frequently during the warmer summer months. The 16 longest durations of fog occurrence exceeded 45 hours in duration. Table 2.3-22 presents fog distribution by the hour of the day at Rockford (1966-1975). Radiative cooling of the earth's surface at night, especially during the summer, produces a larger occurrence of night-time fog. However, the Rockford observations are taken in a river valley, where moisture addition to the atmosphere from the river and wet valley areas is probable. The Byron Station site is located on a plateau with a 145-foot higher elevation. Because of this difference in elevation, fog would be expected to occur less frequently at the Byron site than at Rockford.

2.3.2.1.6 Atmospheric Stability

Onsite temperature difference data between the 30-foot and 250-foot tower levels at the Byron site were used to estimate stability by relation with the temperature lapse rate (change of temperature with height). Monthly and annual Pasquill stability class frequency statistics for the period-of-record January 1974 through December 1976 at the Byron site are presented in Table 2.3-23. Stability persistence data for the Byron site for the same period of record are in Table 2.3-24.

Table 2.3-49 identifies the onsite wind speed and wind direction conditions for the occurrences of extremely stable (G) stability persisting for greater than 10 hours at Byron Station. Also described are the synoptic flow patterns and associated cloud cover in the vicinity of the Byron site.

Examination of the stability data for Byron shows that the slightly stable, or E, stability class occurs 40.54% of the time. The frequencies of occurrence of the stability classes on either side of the neutral (D) and slightly stable (E) classes taper off sharply. Such results are inherent in this classification scheme.

The combination of low wind speeds, a constant wind direction, and a stable atmosphere produces the worst atmospheric diffusion conditions. Listed below is the longest persistence of one wind direction and calm winds occurring during each stability class at the Byron 30-foot level:

Pasquill Stability Class	Longest Persistence In One Wind Direction	Longest Persistence During Calm Winds
A	7 hours (SW,SSW)	2 hours
B	6 hours (E)	1 hour
C	5 hours (NNE,ENE)	1 hour
D	17 hours (WNW)	4 hours
E	18 hours (S)	6 hours
F	9 hours (S)	5 hours
G	9 hours (SSW)	11 hours

Joint frequency distributions of wind direction and wind speed for each Pasquill stability class at the Byron 30-foot and 250-foot levels (1974-1976) are presented in Table 2.3-25 and Table 2.3-26, respectively. This data is used for the long-term diffusion estimates presented in Subsection 2.3.5. For the 30-foot level at Byron, the combination of A stability and calm winds occurred 0.05% of the time; B and calm 0.01%; C and calm 0.02%; D and calm 0.30%; E and calm 0.50%; F and calm 0.51%; and G and calm 0.53%.

Figure 2.3-21 compares the short-term vertical temperature gradient histograms for Byron (1974-1976) and Carroll County (1975-1976). The following data summarized from Figure 2.3-21 compares the Byron frequencies of occurrence (in percent) of each stability class with those of Carroll County:

Stability Class	Byron	Carroll County
A	2.65	4.45
B,C	6.31	4.06
D	33.59	43.74
E	40.54	34.06
F	12.56	11.14
G	4.35	2.56

As indicated previously, there are some differences in the distribution of stability classes between the Byron and Carroll County sites (i.e., the most frequent stability class at Byron is E, 40.54% of the time, while stability Class D is most common at Carroll County, 43.74% of the time.)

However, Figure 2.3-21 shows that the vertical temperature gradient histograms for both sites are similar. Both histograms approximate a Gaussian distribution, with a skew (or tail-out) toward the larger positive (more stable) gradients. The slightly taller and narrower Carroll County peak can be expected since the Carroll County upper temperature measurement level is 80 feet higher than Byron's 250-foot level; the largest temperature variations occur near the ground due to surface heating during the day and radiative cooling at night.

Long-term joint frequency distributions of wind direction and wind speed for each Pasquill stability class at Rockford (1966-1975) are summarized in Table 2.3-27. The stability estimates for Rockford were based on criteria established by Pasquill (Reference 21). The resulting Rockford stability class frequencies (in percent) as compared to the shorter term Byron (1974-1976) are shown below:

Stability Class	Byron	Rockford
A	2.65	0.34
B,C	6.31	13.42
D	33.59	62.28
E	40.54	9.85
F	12.56	9.32
G	4.35	4.40

This long term offsite data shows a relatively high frequency of occurrence of the neutral class D, 62.28%. Significant increases in frequency above the Byron data are seen in the C and D classes; significant decreases are seen in the A and E classes. Estimation of stability classification from surface data, while useful when no other data exist, can lead to significant biases in stability frequency. This bias is generally toward neutral conditions and away from extremes of stability and instability.

Table 2.3-28 presents persistence of Pasquill stability class statistics for Rockford (1966-1975). The neutral stability class D persists for a substantially larger number of consecutive 3-hourly observations than any other stability class. The largest number of consecutive 3-hourly observations of each stable stability class E through G were: five on three occasions for stability class E; five on four occasions for stability class F; and four on ten occasions for stability class G.

2.3.2.2 Potential Influence of the Plant and Its Facilities on Local Meteorology

The heat dissipation facilities at the Byron Station will consist of two natural draft cooling towers and two mechanical draft cooling towers. The natural draft towers will be used to dissipate heat from condenser cooling water and nonessential service water. The mechanical draft cooling towers will be used to dissipate waste heat from the essential service water. Figure 2.1-6 shows the location of the two natural draft and the two mechanical draft cooling towers. Descriptions of the cooling towers along with their specifications and operating parameters are provided in Section 3.4 of the Byron/Braidwood Environmental Report.

Heat rejected to the condenser cooling water and service water will be dissipated primarily by evaporation (latent heat transfer) and convection (sensible heat transfer) to the ambient air drawn into the cooling towers. During these transfer processes, the ambient air drawn into the tower becomes warmer and more moist, and leaves the tower at a higher temperature and usually at a greater relative humidity. A small fraction of the cooling water circulated through the towers is also discharged directly to the atmosphere as drift droplets. It is this rising plume of warm moist air containing draft droplets and its interaction with the environment that cause the meteorological effects of wet cooling tower operation. These effects are divided into several main categories for discussion in this subsection: visible plume effects, drift effects, and other effects, including icing and fog.

Meteorological data gathered at the Byron site for the period March 1974 through February 1977 were available for analysis. Because the recovery of relative humidity data was quite low for some intervals in the period, and because relative humidity is a critical parameter for cooling tower plume modeling, a synthetic year of onsite data was formulated from those months which exhibited the highest relative humidity recoveries. Investigation of Rockford data showed that year-to-year variation of relative humidity was not great in most months for 1974-1977. It therefore appears that the concept of a synthetic year of meteorological data at Byron is generally valid. The months chosen for the final data sample were March through May 1974, June through October 1975, and November 1976 through February 1977.

The final data sample used for all model calculations except drift modelling consisted of hourly values of 250-foot level wind speed, wind direction, dry bulb temperature, wet bulb temperature and relative humidity; and atmospheric stability based on the 30-to-250-foot level temperature gradient. Monthly, seasonal, and annual (synthetic year) three-way joint frequency distributions or wind speed, wind direction, and stability were utilized for drift deposition modeling.

2.3.2.2.1 Visible Plume

The term "visible plume" as used here refers to any cooling tower plume containing condensed water droplets and thus comprising an opaque cloud that would be visible to an observer given adequate light and an absence of intervening obstructions to vision. Frequently, therefore, a visible plume will not be truly visible from the ground because of nighttime darkness or natural fog and clouds.

2.3.2.2.1.1 Natural Draft Cooling Towers

2.3.2.2.1.1.1 Temporal and Spatial Distribution of Plumes

Tables 2.3-28a and 2.3-28b summarize results of model computations for the Byron natural draft towers based on one year of meteorological data. The data are all for full-load operation and therefore overestimate the frequencies that will actually occur. The median plume length is 280 meters for summer and 5500 meters for winter. Note that 8.5% of all plumes (19.2% in winter) essentially extend indefinitely, which should be interpreted as stating that they merge with existing natural cloud layers.

Plume occurrences in Tables 2.3-28a and 2.3-28b are summed over all wind directions. A table entry thus means, for example, that 75% of the time over a year a visible plume will extend 200 meters or further from either cooling tower. The direction of the plume will vary with wind direction, as discussed in the following.

Table 2.3-28b indicates that most cooling tower plumes will occur between the tower top (151 meters) and a height of 500 meters. However, visible plumes may occasionally rise as high as 3,000 meters (10,000 feet) above the ground. According to the model results, very high plumes are most likely in summer, when winds are often light and the atmosphere unstable, and in winter when plume buoyancy is greatest. The median plume height is less than 500 meters however, and fewer than 10% of all plumes are expected to rise above 1,000 meters.

Figures 2.3-21a through 2.3-21d show the geographical distribution of visible plumes for each season. The isopleths represent the number of hours per season that visible plumes can be expected to occur over a given point. The data used to generate the maps were adjusted for an expected seasonal capacity factor of 55% for the spring and fall seasons and 75% for the summer and winter seasons.

The figures generally show the expected distribution of plume lengths and directions. Note that the highest frequency of plumes in summer is to the north, due to prevailing southerly winds. The number and length of plumes are much greater in winter, as expected.

The most significant fact shown by the figures is that, despite the frequency of long plumes indicated by Table 2.3-28a, the frequency of plumes passing over any given point represents a quite small fraction of the time. Even in winter, all points more than 1 mile from the cooling towers will have fewer than 160 hours per season, or 7.5% of the time, that plumes reach them. In Byron, 4 miles from the cooling towers, the average incidence of overhead plumes is indicated to be 22 hours per year.

2.3.2.2.1.1.2 <u>Visible Plume Impact Assessment</u>

The primary impact of the visible plumes from Byron natural draft towers will be visual. Plumes will often be visible for large distances because of their size and the exposed, elevated site location. Of course, the natural draft towers themselves have a substantial visual impact, and plumes will add to this. Visible plume impacts at night, in naturally cloudy weather, and in summer will be minimal. Effects will be greatest on clear winter days, particularly on cold, calm winter mornings, when visible plumes may tower several thousand feet above the station.

Shadowing on the ground and decrease of sunlight due to visible plumes aloft will be negligible compared to natural year-to-year variation in cloudiness. The hours of visible plume occurrence as shown in Figures 2.3-21a through 2.3-21d can be taken as highly conservative estimates of hours of shadowing. It must be considered that many visible plumes will occur at night and in cloudy weather when no shadowing will occur. During the summer growing season, sunlight may be reduced from 5 to 25 hours per season for offsite areas surrounding the plant property.

Visible plumes aloft will not interfere with operations at area airports. Expected annual frequencies are approximately 36 hours per year in the sector of Davis and Stukenberg private airports northwest of the site. These hours include times of natural clouds. Plumes will be at altitudes of at least 300 meters (1000 feet) above the ground and limited in horizontal extent.

As previously indicated, the natural draft cooling towers will not cause fog at ground level. Modeling results and experience both show that plumes from tall natural draft towers do not descend to ground level after emission from the towers.

2.3.2.2.1.2 Mechanical Draft Cooling Towers

In normal operation, the essential service water cooling towers will dissipate a very small quantity of heat in comparison to the natural draft towers (0.24×10^9 Btu/hr versus 15.8×10^9 Btu/hr). Visible plumes and atmospheric impacts due to operation of mechanical draft towers will be correspondingly small compared with those of the natural draft towers. Because of their small size, impacts of the mechanical draft towers have been assessed based on analyses previously performed and experience with mechanical draft towers in similar climates.

Visible plumes are expected to range from near-zero length and height in summer to about 500 meters in length and 200 meters in height during extremely cold weather. It is estimated that visible plumes will exceed 100 meters in length 10% to 15% of the time.

2.3.2.2.2 Impacts of Drift

2.3.2.2.2.1 Natural Draft Cooling Towers

A small fraction of the circulating water from the Byron cooling system will be discharged directly to the atmosphere as drift droplets. The drift rate for Byron natural draft towers is 0.002%, or approximately 13 gal/min per tower at full load.

By discharge of drift, the cooling towers will release a small quantity of dissolved solids to the atmosphere. The estimated maximum solids concentration in drift water will be approximately 1031 mg/liter. Solids constituents are primarily calcium and magnesium sulfates, plus a smaller amount of sodium chloride, silica, and phosphate. Maximum emission rate of drift solids, for .002% total drift, will be 0.1 lb/min per tower.

2.3.2.2.2.1.1 Solids Deposition

Figure 2.3-21e shows the calculated annual distribution of deposited solids, based on two towers at 65% capacity factor. The figure shows deposition rate in pounds per acre-month as a function of direction and distance from the cooling towers. Maximum average deposition rate is 2.5 lb/acre-month, which is predicted to occur 700 meters (.43 mile) east of the towers. Values decrease in all directions beyond this distance, which represents fallout of the largest drift droplets in stable stratification and light wind.

Heaviest deposition is north and east of the towers due to prevailing wind directions. In general, deposition will be 0.5 to 2.5 lb/acre-month within 1 mile of the towers, and will decrease to the order of 0.1 lb/acre-month by 3 miles distance.

The data in Figure 2.3-21e are annual averages, based on expected capacity factor and annual distribution of meteorological parameters. Short-term deposition rates will be higher during those times when the wind is blowing toward a particular receptor and the plant is operating at capacity. The maximum local rate could be on the order of 3 lb/acre-day (90 lb/acre-mon) within 0.5 mile of the towers. This rate of deposition will only persist for the interval (usually several hours or less) that the wind remains oriented in a given direction, and stable stratification, high humidity, and low wind speeds persist.

Predicted drift deposition will have very little impact on lands, waters and vegetation surrounding the plant. Typical background quantities of dissolved solids deposited by rainfall range from 2 to 15 lb/acre-mon in continental areas. Rainfall analyses from the Chicago area indicate background deposition rates of 5 to 15 lb/acre-mon; data from central New York State yield rates of 4 to 8 lb/acre-mon (Reference 25). A study of salt damage to corn and soybeans (Reference 26) indicated that the first evidence of any damage occurs with salt applications of 7 lb/acre-mon above background for 8 weeks. The maximum expected deposition from Byron cooling towers is thus below typical background and well below rates shown to produce adverse effects.

2.3.2.2.2.1.2 Drift Precipitation

Although the primary impact of drift is solids deposition, fallout of liquid water will also occur to the extent that the larger drift droplets reach the ground before evaporation. A highly conservative estimate of drift precipitation can be obtained by assuming no evaporation. Under that assumption, the maximum long-term average fallout (precipitation) rate would be 0.01 inch of water per month. Maximum short-term rate would be approximately 1 cm/month (0.4 in./month) or 0.03 cm/day. These quantities of precipitation from drift fallout are negligible compared to natural precipitation.

When the ground temperature is below freezing, drift fallout can produce icing on exposed surfaces. The maximum short-term drift fallout rate for a given point, if it persisted for 24 hours, would produce an ice accumulation of less than 1 millimeter (0.04 inch) thickness. This extent of icing is quite unlikely, because wind direction, wind speed and stability seldom remain constant for more than a few hours. One millimeter of ice is too little to cause any significant vegetation damage or transportation hazard.

Experience indicates that some droplet fallout and icing occasionally occur near the base of natural draft towers as a result of circulating water blown out of the fill by strong surface winds. Occurrence of this phenomenon is not readily predictable, since it depends on tower construction, local wind exposure, and the aerodynamics of surface flow patterns around the towers and other plant structures. While this "blow through" can produce significant icing during severe winter weather, the effects would be restricted to the immediate vicinity of the towers (within several hundred feet).

2.3.2.2.2.1.3 Airborne Solids

When drift evaporates, the resulting solid particles remain in the air until they are removed by sedimentation, impaction on vegetation, or precipitation washout. These drift solids constitute a local source of sulfates and other particulate matter to the ambient air.

Maximum 24-hour concentrations of drift solids will be found about 2 km downwind of the towers and could reach values on the order of 1 to 2 $\mu g/m^3$. The National Secondary Ambient Air Quality Standard for suspended particulate matter is 150 $\mu g/m^3$. Thus, the cooling tower drift contribution to ambient particulate concentrations will be quite small and generally undetectable.

2.3.2.2.2.2 Mechanical Draft Cooling Towers

Total drift from the two essential service water cooling towers will be approximately 2.4 gpm under normal operating conditions. This is less than 10% of the drift from the natural draft towers. Drift fallout will be restricted to areas much closer to the tower than in the case of the natural draft towers because of the lower emission height. Offsite solids deposition, precipitation, and particulate concentrations will be considerably less than from the primary cooling System towers and will be negligible. Drift fallout within several hundred feet of mechanical draft towers can be heavy enough to produce ice accumulations in winter and some light solids deposition. The location of the service water towers on the Byron site is such that these effects will have no serious impact on plant operations.

2.3.2.2.3 Other Cooling Tower Effects

2.3.2.2.3.1 Influences on Climate

Emissions from large cooling towers have been observed to produce cumulus and stratocumulus clouds (References 27 and 28). Light snowfall, up to a maximum of 1 inch, has also been reported (Reference 28). When clouds or simply long visible plumes occur, some local shadowing of ground areas is produced, as discussed in Subsection 2.3.2.2.1.1.2. Observations of these phenomena for towers in climates similar to that of the Byron Station site indicate that these phenomena occur rarely enough and over sufficiently small areas that they have caused no measurable changes in basic climatological statistics.

Martin (Reference 29) and Moore (Reference 30) have reported on climatological studies around stations in England with natural draft cooling towers and have summarized long-term experience. They concluded that there are no observable changes in rainfall, sunshine, fog, humidity, or temperature from the operation of the towers.

Despite the large size and appreciable number of towers now operating in the United States, there have been no reports of meaningful environmental effects. A number of studies are now in progress to determine whether emissions from a number of large cooling towers serving "energy parks" with 10,000- to 50,000-MW generating capacity might produce significant weather effects. However, it is generally accepted (Reference 31) that atmospheric effects resulting from sources with current heat dissipation rates are not serious problems for properly designed power stations at appropriate sites.

There is no reason to expect unusual or different impacts at Byron. It is concluded, on the basis of experience with similar towers and sites, that no detectable changes in the normal amounts of cloudiness, precipitation, and sunshine, or of temperature or humidity will occur due to operation of the Byron cooling towers. There will be some occasions, however, when small clouds will be induced downwind of the towers or when existing cloud layers will be augmented by the cooling tower emissions. These occasions are most likely in winter, and it is possible that light snowfall could also be initiated from the tower plumes on a few occasions each winter.

2.3.2.2.3.2 Icing and Fog

The deposition of ice from cooling tower drift and its impact are discussed in Subsection 2.3.2.2.3.1.3. It was concluded that ice from drift fallout is not likely to occur in magnitudes sufficient to cause any significant impacts. Ice can also form when the visible plumes themselves contact cold solid surfaces.

Supercooled cloud droplets in the plume freeze on contact, forming low-density opaque ice called rime.

Since plumes from the Byron Station natural draft cooling towers will not contact the ground or plant structures, rime icing is not expected. Similarly, fog will not occur. Observations at five natural draft tower sites in the Ohio Valley (Reference 28) led to the conclusion that "the tower plumes themselves never caused fog" and "it (icing) was never observed." The observations included 343 cases between 1973 and 1976.

Rime icing is sometimes extensive near large mechanical draft cooling towers where downwash brings visible plumes in contact with the ground. Rime icing due to mechanical draft cooling towers at the Byron Station site will have little impact because it will be primarily restricted to open areas within 100 meters east and southeast of the towers.

When wind speeds are high, the Byron service water towers will cause ground fog in the lee of the towers. This is a downwash effect unique to mechanical draft towers. Fog will occur only within approximately 100 meters (328 feet) of the towers, and only with strong wind, high humidity, and relatively low temperature. Annual occurrence will be approximately 300 hr/yr at a distance of 50 meters. Because of tower orientation and site climate, fog will usually occur to the east and southeast of the towers. There are no plant facilities in those directions that will be adversely affected by these occasional fog occurrences. The plant access road is, at closest, 100 meters south-southeast, which will normally be beyond any cooling-tower fog that occurs in that direction.

2.3.2.2.3.3 Interaction with Atmosphere Constituents

The suggestion is occasionally made that cooling tower plumes could be involved in interactions with pollutants in the atmosphere (principally SO) so as to intensify pollution problems. Sulfur dioxide can react with water to form acid droplets and ultimately sulfate particulates. The closest source of significant SO emissions is Rockford. Cooling tower plumes from Byron will pass over Rockford less than 10% of the time (based on the onsite wind rose), and at the distance of Rockford, plumes will normally be highly diluted and all liquid water will have evaporated.

It is important to recognize that water vapor contained in a cooling tower plume is a natural constituent of the atmosphere and is already present in large quantities. Liquid water droplets are also present much of the time in the atmosphere as natural clouds. At distances beyond several kilometers from the cooling towers, moisture contributions from the towers will represent a very small perturbation to naturally occurring water vapor concentration. Thus it is unlikely that the cooling tower plume could appreciably alter distant atmospheric reactions involving water. There are no emissions from Byron Station that would be expected to interact with cooling tower water at close range where visible plumes regularly occur. Therefore, Byron cooling tower plumes are not expected to have significant synergistic effects with other pollutants or constituents of the atmosphere.

2.3.2.3 Topographical Description

Figure 2.3-22 is a topographic map showing the area surrounding the Byron Station. Figure 2.3-23 shows topographic cross sections in each of the 16 compass-point directions radiating from the site. It can be seen that the plant, at an elevation of approximately 865 feet above MSL, is at one of the highest points within a 5-mile radius. Terrain in the immediate vicinity of the plant falls off in all directions except to the south and southeast. The primary topographic feature in the area is the Rock River at an elevation just under 700 feet. The slope from higher terrain at the plant to the river is generally gradual in all directions. Terrain within a 50-mile radius of the Byron site is generally level, imposing little obstruction to mesoscale air flow.

No significant effect of topography on atmospheric dispersion is anticipated due to the modest relief of the area. Accordingly, no topographic factors have been included in dispersion calculations (Subsections 2.3.4 and 2.3.5). Because the plant site is generally above surrounding terrain, any topographic influences that do exist are expected to improve dispersion and lead to lower ground-level dilution factors than those calculated for a flat-terrain assumption.

Cool air drainage downslope to the Rock River valley is likely under stable nighttime conditions. The effect will not be pronounced due to the modest slopes, but should be detectable at times of nearly calm general winds. However, the plant site is on a plateau where drainage currents originate. Therefore, airflow will not be dominated by drainage even on nights favorable for such winds. Dispersion of plant effluents under the conditions of downslope winds should be adequately described by the stable stratification and light wind conditions.

2.3.3 Onsite Meteorological Measurements Program

The meteorological tower is located as shown on Figure 2.1-6.

The meteorological measurements program consists of monitoring wind direction, wind speed, temperature, and precipitation. Two methods of determining atmospheric stability used are:

a. delta T (vertical temperature difference) is the principal method, and

b. sigma theta (standard deviation of the horizontal wind direction) is available for use when delta T is not available.

These data, referenced in ANSI/ANS 2.5 (1984), are used to determine the meteorological conditions prevailing at the plant site. The meteorological program includes site-specific information on instrumentation and calibration procedures. The meteorological program meets the requirements of the Offsite Dose Calculation Manual.

The meteorological tower is equipped with instrumentation that conforms with the system accuracy recommendations in Regulatory Guide 1.23 and ANSI/ANS 2.5 (1984). The equipment is placed on booms oriented into the generally prevailing wind at the site. Equipment signals are transmitted to an instrument shack with controlled environmental conditions. The shack at the base of the tower houses the recording equipment, signal conditioners, etc., used to process and retransmit the data to the end-point users.

Recorded meteorological data are used to generate wind roses and to provide estimates of airborne concentrations of gaseous effluents and projected offsite radiation dose. Instrument calibrations and data consistency evaluations are performed routinely to ensure maximum data integrity. The data recovery objective is to attain better than 90% from each measuring and recording system. Data storage and records retention are also maintained in compliance with ANSI/ANS 2.5 (1984)

Pages 2.3-34 through 2.3-38 have been deleted intentionally

2.3.4 Short-Term (Accident) Diffusion Estimates

2.3.4.1 Objective

Conservative estimates of the local atmospheric dilution factors (χ/Q) and their 5% and 50% probability levels for the Byron Station site have been made. These estimates were made for the minimum exclusion area boundary (the minimum actual site boundary) and the outer boundary of the low population zone (LPZ) for each of the 16 cardinal directions. Estimates were made for time periods from 1 hour to 2 hours for the minimum exclusion area boundary and from 8 hours up to 30 days for the outer boundary of LPZ, utilizing onsite meteorological data (36-month period of onsite data recorded from January 1, 1974 through December 31, 1976).

The effects of spatial and temporal variations in airflow in the region of the Byron site are not described by the constant mean wind direction model (used in calculating the Byron χ/Q values) since the model uses single-station meteorological data to represent diffusion conditions within the vicinity of a site. Regulatory Guide 1.111 recommends that if a constant mean wind direction model is used, airflow characteristics within 50 miles of the site should be examined to determine the spatial and temporal variations of atmospheric transport and diffusion.

Recirculation of airflow and wind directional biases during periods of prolonged atmospheric stagnation are the primary variations in atmospheric transport and diffusion conditions to be considered for the Byron site. Airflow is not inhibited by topography in the vicinity of Byron due to the relatively flat nature of the terrain. A detailed topographic description is presented in Subsection 2.3.2.3. Regional airflow is dominated by large-scale (synoptic) weather patterns. A summary of these climatological patterns is in Subsection 2.3.1. Figures 2.3-24 through 2.3-26 are wind roses for locations near the Byron site. The similarity of wind direction distribution from location to location indicate the common wind regime representative of the area.

A combination of low wind speeds, a constant wind direction, and a stable atmosphere produces the worst atmospheric dispersion conditions. Under very stable conditions (Pasquill "G" stability), the longest persistence of one wind direction is 9 hours (south-southwest). For calm conditions, the longest persistence of G stability is 11 hours (see Subsection 2.3.2.1).

Based on the regional airflow characteristics for the Byron site, it is concluded that the constant mean wind direction model which utilizes single-station meteorological data is an acceptable method of calculating χ/Q values for the Byron site.

A comparison of results from a variable trajectory model and a constant mean wind direction model at a site with meteorological and topographic similarities to the Byron site indicated that the constant mean wind direction model produced χ/Q values about a factor of four lower than the variable trajectory model for certain locations. It should be noted, however, that the variable trajectory model χ/Q values are extremely conservative in this particular case as a result of two important input conditions. These conditions are noted in a memorandum from G. E. Start (Deputy Chief, ARLFRO, ERL, Idaho Falls, Idaho) to C. R. Dickson (Chief, ARLFRO, ERL, Idaho Falls, Idaho) dated May 3, 1976, and are stated in the following paragraphs.

Single-station meteorological data was the only data used as input. The primary advantage of using the variable trajectory model, the ability to use regional data, was not a factor in the χ/Q calculations. Therefore, a partial explanation of the difference in the results from the two models is related to the difference in the diffusion equations used rather than the spatial variation in meteorology.

A constant mixing height of 1000 feet was assumed in the variable trajectory model for the Byron case. Since the constant mean wind direction model does not consider mixing height limitations, it is apparent that the use of this relatively low-level, constant mixing height in the variable trajectory model was partially responsible for the conservatism of the χ/Q values.

For comparative purposes, it is recommended that the NRC use a variable trajectory model with regional data from several locations near Byron. It is further suggested the same mixing height assumptions be used in both the variable trajectory model and the constant mean wind direction model. The results of this type of comparison will likely yield a more realistic evaluation of χ/Q values from each model.

Estimates of atmospheric diffusion (χ/Q) at the Exclusion Area Boundary (EAB) and the outer boundary of the Low Population Zone (LPZ), calculated for the regulated short-term (accident) time-averaging periods of 0-2 hrs, 2-8 hrs, 8-24 hrs, 1-4 days, and 4-30 days were also performed in support of Alternative Source Term (AST) implementation.

2.3.4.2 Calculations (For use with TID-14844 based dose analyses)

Calculations of the short-term atmospheric dilution factors (χ/Q) for the Byron Station site were performed using Gaussian plume diffusion models for ground-level concentrations resulting from a continuously emitting source. To be conservative the effluent release level was assumed to be at ground level and total reflection of the plume was assumed to take place at the ground surface; i.e., there is no deposition or reaction at the surface.

Hourly χ/Q values were calculated using a centerline diffusion model for time periods up to 8 hours and a sector-average diffusion model for time periods longer than 8 hours. A building wake correction factor that did not exceed a maximum of 3.0 was used in the centerline model to account for additional dilution due to wake effect of reactor building. No credit was given for additional building wake dilution for the sector-average model. Mathematical expressions of the models are as follows:

a. for time periods up to 8 hours,

$$\chi/Q = \frac{1}{\pi\, u\, \sigma_y\, \sigma_z + cAu} \qquad (2.3\text{-}4)$$

$$= \frac{1}{D_b}\,\frac{1}{\pi\, u\, \sigma_y\, \sigma_z}$$

b. for time periods greater than 8 hours

$$\chi/Q = \frac{2.032}{\sigma_z\, u\, X} \qquad (2.3\text{-}5)$$

where:

χ/Q = ground-level relative concentration (sec/m^3);

u = mean wind speed (m/sec);

σ_y = horizontal diffusion parameter (m);

σ_z = vertical diffusion parameter (m);

c = empirical building shape factor;

A = reactor building minimum cross section (m^2);

D_b = building wake correction factor $1 + \left(\frac{CA}{\pi\sigma_y\sigma_z}\right)$;

X = distance from release point to receptor; and

2.032= $\sqrt{2}/\pi$ divided by width in radians of a 22.5° sector.

Meteorological data input used were concurrent hourly mean values of wind speed and wind direction measured at the 30-foot level and Pasquill stability class determined by the measured vertical temperature difference (ΔT) between 30- and 250-foot levels.

Both σ_y and σ_z are functions of downwind distance from the point of release to a receptor, and the Pasquill stability class. The numerical values of σ_y and σ_z for Pasquill stability classes A through F were digitized from Gifford's graphs (Reference 21).

The values of σ_z for Pasquill stability classes A and B have been cut off at 1000 meters. The values of σ_y and σ_z for Pasquill stability class G were determined from σ_y and σ_z for Pasquill stability class F using the following equations:

$$\sigma_y(G) = \frac{2\,\sigma_y(F)}{3}$$

$$\sigma_z(G) = \frac{3\,\sigma_z(F)}{5} \qquad (2.3\text{-}6)$$

A building shape factor of 0.5 and building minimum cross section of 2700 m^2 were used to determine the building wake correction factor.

For calm wind conditions a wind speed of 0.17 m/sec, one-half of the threshold speed, was assigned, and computed χ/Q values for calm wind conditions were assigned to the wind direction for the previous hour.

From these hourly χ/Q values, mean values were computed for sliding time period "windows" of 2, 8, 16, 72, and 624 hours for each of 16 cardinal directions. These intervals correspond to accident time periods of 0-1 hour, 0-2 hours, 0-8 hours, 8-24 hours, 1-4 days, and 4-30 days. For each time interval used, the maximum χ/Q value in each sector was identified and the mean χ/Q in each was computed. The cumulative frequency distribution for each of the individual time periods was then prepared, and from this distribution the fifth and fiftieth percentile χ/Q values were estimated for each of the 16 cardinal sectors.

Cumulative frequency distributions of χ/Q values and 5% and 50% probability levels of χ/Q at the minimum exclusion area boundary distance of 445 m (minimum actual site boundary distance) are presented in Tables 2.3-29 through 2.3-31 for accident time periods of 0-1 hour and 0-2 hours. Cumulative frequency distributions of χ/Q values and the maximum, 5%, and 50% probability levels of χ/Q values at the outer boundary of LPZ are presented in Tables 2.3-32 through 2.3-38 for accident time periods of 0-8, 8-24 hours, 1-4 days, and 4 to 30 days.

Provided in Table 2.3-54 are the minimum exclusion area boundary (MEAB) distances for the Byron site. The MEAB distances were derived using methodology that was used prior to the issuance of Regulatory Guide 1.145 and is considered acceptable by the NRC.

2.3.5 Long-Term (Routine) Diffusion Estimates (For TID-14844 based dose analyses)

2.3.5.1 Objective (For TID-14844 based dose analyses)

Realistic estimates of annual average atmospheric dilution factors (χ/Q) for effluents released routinely from the 200-foot Byron vent stack have been made. These estimates are made for site boundary distances and for various distances out to 50 miles (80.5 km) for each of the 16 cardinal directions, utilizing onsite meteorological data (36-month period of onsite data recorded from January 1, 1974 through December 31, 1976).

2.3.5.2 Calculations (For TID-14844 based dose analyses)

Annual average atmospheric dilution factors (χ/Q) for the Byron Station site are calculated using the sector-average Gaussian plume diffusion model (constant mean wind direction model) modified to account for various modes of effluent release according to the recommendations of Regulatory Guide 1.111, Methods for Estimating Atmospheric Transport and Dispersion of Gaseous Effluents in Routine Releases From LWRs (Revision 1, 7/77).

2.3.5.2.1 Joint Frequency Distribution of Wind Direction Wind Speed, and Stability (For TID-14844 based dose analyses)

The effluents released from the Byron Station vent stack may be considered either elevated or ground-level releases, depending on the ratio of the vertical exit velocity of the stack discharge to the horizontal wind speed. To accommodate this variation and to utilize the appropriate meteorological data obtained at the 30-foot and 250-foot levels of the Byron onsite meteorological tower, a composite joint frequency distribution of wind speed, wind direction, and stability class for each of the elevated or ground-level release conditions is determined from an hour-by-hour scan of wind speed data recorded at both levels of the meteorological tower.

The stack height level wind speed is chosen as the most representative for determining the release condition. The power law, as expressed below, is used to extrapolate the hourly average 250-foot level wind speed to the 200-foot stack height level (Reference 22):

$$u = u_{250} \left(\frac{200}{250}\right) p \qquad (2.3\text{-}7)$$

where:

u	=	wind speed extrapolated to the 200-feet stack height level;
u_{250}	=	measured wind speed at the 250-foot meteorological tower level; and
p	=	exponent dependent on atmospheric stability class as follows:

Stability Class	A	B	C	D	E	F	G
p	0.10	0.15	0.20	0.25	0.25	0.30	0.30

The exponential factors (p), as described in Reference 22, were empirically derived from a meteorological tower experiment performed at the Brookhaven National Laboratory. In the experiment, two sets of 500 wind and temperature records taken at several tower levels were utilized. Atmospheric stability was determined by taking the temperature difference between two tower levels (ΔT). The exponential factors (p) were derived using the formula $u = u_1 (Z/Z_1)^p$ for various degrees of atmospheric stability. U is the wind speed at height Z, and U_1 is the wind speed at height Z_1.

To determine if an elevated or ground-level release condition exists, a ratio, V_R, of the vertical exit velocity of the effluent plume to horizontal wind speed at the stack height level is defined:

$$V_R = \frac{Wo}{u} \qquad (2.3\text{-}8)$$

where

W_o	=	vertical exit velocity of the effluent plume; and
u	=	stack height wind speed

The ratio V_R is then used in the following equations to define an entrainment coefficient, E_t (Reference 23):

$$E_t = 1.00 \text{ for } V_R<1.0$$

$$E_t = 2.58 - 1.58\ V_R \text{ for } 1\le V_R\le 1.5$$

$$E_t = 0.3 - 0.06\ V_R \text{ for } 1.5<V_R<5.0$$

$$E_t = 0 \text{ for } V_R\ge 5.0 \qquad (2.3\text{-}9)$$

For the hour being scanned, the release is considered an elevated release $(1-E_t) \times 100\%$ of the time and a ground level release $E_t \times 100\%$ of the time. The total time duration of the elevated release, t_e, is then calculated as:

$$t_e = (1-E_t) \times 1 \text{ hour} \qquad (2.3\text{-}10)$$

and the total time duration of the ground level release, t_g, is calculated as:

$$t_g = E_t \times 1 \text{ hour} \qquad (2.3\text{-}11)$$

The elevated and ground level joint frequencies $(F_{isd})_e$ and $(F_{isd})_g$ for each stability class i, wind speed s, and wind direction d are determined as follows:

$$(F_{isd})_e = \frac{\sum_{j=1}^{T} (t_{isd})_{e,j}}{T} \qquad (2.3\text{-}12)$$

$$(F_{isd})_g = \frac{\sum_{j=1}^{T} (t_{isd})_{g,j}}{T} \qquad (2.3\text{-}13)$$

where:

$(F_{isd})_e$ = joint frequency of stability class i, wind speed class s, and wind sector d applicable to the elevated release condition as derived from the 250-foot level meteorological data with the wind speed extrapolated to 200-foot stack height level,

$(F_{isd})_g$ = joint frequency of stability class i, wind speed class s, and wind sector d, applicable to the ground-level release condition as derived from the 30-foot level meteorological data;

$(t_{isd})_{e,j}$ = time duration of wind speed class s, wind sector d, and stability class i at the 200-foot stack height level during hour j;

$(t_{isd})_{g,j}$ = time duration of wind speed class s, wind sector d, and stability class i at the 30-foot meteorological tower level during hour j; and

T = total number of scanned hours with valid data.

It is noted that:

$$\sum_{i,s,d} (F_{isd})_e + \sum_{i,s,d} (F_{isd})_g = 1 \qquad (2.3\text{-}14)$$

2.3.5.2.2 Effective Released Height (For TID-14844 based dose analyses)

For an elevated release, the effective release height, h_e, is defined blow:

$$h_e = h_s + h_{pr} - C \qquad (2.3\text{-}15)$$

where:

h_e = effective release height;

h_s = physical stack height

h_{pr} = plume rise (defined below); and

C = correction for low exit velocity (defined below).

Because of the modest relief surrounding the Byron Station site, receptor terrain heights have not been considered in determining the effective release height (see Subsection 2.3.2.3). The plume rise, h_{pr}, is calculated from the following momentum plume rise equations (Reference 24):

For neutral or unstable conditions, the smaller value from the two following equations is used:

$$h_{pr} = 1.44 \left(\frac{W_o}{u}\right)^{\frac{2}{3}} x \left(\frac{X}{D}\right)^{\frac{1}{3}} x\ D \qquad (2.3\text{-}16)$$

or:

$$h_{pr} = 3.0 \left(\frac{W_o}{u}\right) x\ D \qquad (2.3\text{-}17)$$

where:

h_{pr} = plume rise;

W_o = vertical exit velocity of effluent plume;

X = downwind distance from release point;

u = stack height wind speed; and

D = internal stack diameter.

For stable conditions, the results from Equations 2.3-16 and Equation 2.3-17 are compared with the results from the following two equations, and the smallest among the values obtained from Equations 2.3-16 through 2.3-19 are then used:

$$h_{pr} = 4\left(\frac{F_m}{S}\right)^{\frac{1}{4}} \qquad (2.3\text{-}18)$$

$$h_{pr} = 1.5\left(\frac{F_m}{u}\right)^{\frac{1}{3}} \times S^{-\frac{1}{6}} \qquad (2.3\text{-}19)$$

where:

h_{pr} = plume rise;

F_m = momentum flux parameter = $W_o^{\,2}\left(\frac{D}{2}\right)^2$;

S = a stability parameter = $\frac{g}{T}\frac{\partial\theta}{\partial Z}$

where:

g = acceleration of gravity;

T = ambient air temperature; and

$\frac{\partial\theta}{\partial Z}$ = vertical potential temperature gradient.

In the calculations S was defined as 8.7×10^{-4} sec^{-2} for E stability, 1.75×10^{-3} sec^{-2} for F stability, and 2.45×10^{-3} sec^{-2} for G stability.

When the vertical exit velocity is less than 1.5 times the horizontal wind speed, a correction, for stack downwash, C, is subtracted from the effective stack height as shown in Equation 2.3-15 (Reference 23):

$$C = 3\,(1.5 - W_o/u)\,D \qquad (2.3\text{-}20)$$

where:

C = downwash correction factor;

D = inside diameter of the stack;

u = wind speed at stack height; and

W_o = vertical exit velocity of the plume.

For ground-level releases, the effective release height of all times is zero ($h_e = _o$).

2.3.5.2.3 Annual Average Atmospheric Dilution Factor (For TID-14844 based dose analyses)

Using the joint frequency distributions developed in Subsection 2.3.5.2.1, the sector-averaged dispersion equations are used to calculate annual average dispersion factors for location of interest. Equation 2.3-21, given below, is used to calculate the dispersion factor for elevated release conditions:

$$(\chi/Q)_{de} = \frac{1}{B} \sqrt{\frac{2}{\pi}} \quad \Sigma\, i\, \Sigma\, s\, \frac{(F_{isd})_e \exp-\left[\frac{h_e^2}{2\,\sigma^2_{zi}}\right]}{\sigma_{zi}\, u_s\, X} \qquad (2.3\text{-}21)$$

where:

$(\chi/Q)_{de}$ = atmospheric dilution factor (sec/m^3) at a distance X in downwind sector of wind direction d for elevated release conditions;

B = sector width for 22.5° sector = 0.3927 radians;

$(F_{isd})_e$ = joint frequency of stability class i, wind speed class s, and wind sector d, applicable to the elevated release condition;

h_e = effective release height;

σ_{zi} = vertical standard deviation of contaminant concentration at distance X for stability class i (Reference 21);

X = downwind distance from release point; and

u_s = stack height wind speed class s.

For ground level release the following equation is used:

$$(\chi/Q)_{dg} = \frac{1}{B} \sqrt{\frac{2}{\pi}} \quad \Sigma\, i\, \Sigma\, s\, \frac{(F_{isd})_g}{\sigma_{zi} * u_s\, X} \qquad (2.3\text{-}22)$$

where:

$(\chi/Q)_{dg}$ = atmospheric dilution factor (sec/m^3), at a distance X in downwind sector of wind direction d for ground-level release conditions:

B = sector width for 22.5° sector = 0.3297 radians;

$(F_{isd})_g$ = joint frequency of stability class i, wind speed class s, and wind sector d applicable to the ground level release condition;

h_e = effective release height;

σ_{zi}* = vertical standard deviation of contaminant concentration at a distance X for stability class i, corrected for additional dispersion within the reactor building cavity (Reference 21)

= $(\sigma_{zi}^2 + CA)^{1/2} \leq \sigma_{zi} \times \sqrt{3}$

where:

C = building shape factor = 0.5; and

A = minimum cross sectional area of containment building = 2700 m^2;

X = downwind distance from release point; and

u = 30-foot level wind speed class s.

The values of the $(\chi/Q)_{de}$ and $(\chi/Q)_{dg}$ calculated at each downwind distance are added together to give the total annual average dispersion factor, $(\chi/Q)_d$, at that distance:

$$(\chi/Q)_d = (\chi/Q)_{de} + (\chi/Q)_{dg}$$

Annual average χ/Q for the 200-foot Byron vent stack at the site boundary distances and at various distances out to 50 miles (80.5 km) are presented in Tables 2.3-39 and 2.3-40.

The minimum containment building cross-sectional area for the Byron reactor containment building was approximated using the dimensions illustrated in Drawing M-14. The containment is cylindrical in shape with a domed roof. The building height is 60.7 meters and the building diameter is 45.7 meters.

2.3.6 Short-term (Accident) Diffusion Estimates (Alternative Source Term χ/Q Analysis)

2.3.6.1 Objective

Estimates of atmospheric diffusion (χ/Q) at the Exclusion Area Boundary (EAB) and the outer boundary of the Low Population Zone (LPZ) are calculated for the regulated short-term (accident) time-averaging periods of 0-2 hrs, 2-8 hrs, 8-24 hrs, 1-4 days, and 4-30 days.

2.3.6.2 Meteorological Data

The Byron onsite meteorological tower database for the five-year period, 1994-1998, was applied in the modeling analyses. Wind speed and direction taken at 30 ft and 250 ft and the vertical temperature difference data measured between 250 ft and 30 ft were utilized. "Calm" wind speeds were assigned a value of 0.4 mph (1/2 the instrument threshold starting speed value). The combined data recovery of wind speed, wind direction, and stability data exceeded the RG 1.23 (Reference 35) goal of 90 percent for each of the 5 years (1994 through 1998).

2.3.6.3 Calculation of χ/Q at the EAB and LPZ

The calculation of χ/Q at the EAB (i.e., 445 m) for postulated releases from the Unit 1 and Unit 2 outer Containment Wall, and at the LPZ (4828 m) for a postulated release from the midpoint between the two reactors, utilized the NRC-recommended model PAVAN (Reference 33), which implements Regulatory Guide 1.145 methodology. These releases do not qualify as "elevated releases" as defined by Regulatory Guide 1.145 (Reference 34); therefore, they were modeled as "ground" type releases.

The calculation of χ/Q at the EAB and LPZ by PAVAN in accordance with Regulatory Guide 1.145 for ground-level releases, is based on the following equations:

$$\chi/Q = \frac{1}{\overline{U}_{10}\left(\pi\sigma_y\sigma_z + A/2\right)} \qquad (2.3.6\text{-}1)$$

$$\chi/Q = \frac{1}{\overline{U}_{10}\left(3\pi\sigma_y\sigma_z\right)} \qquad (2.3.6\text{-}2)$$

$$\chi/Q = \frac{1}{\overline{U}_{10}\pi\Sigma_y\sigma_z} \qquad (2.3.6\text{-}3)$$

where:

χ/Q is relative concentration, in sec/m^3.

π is 3.14159.

$\overline{U}_{10}$ is wind speed at 10 meters above plant grade, in m/sec.

σ_y is lateral plume spread, in meters, a function of atmospheric stability and distance.

σ_z is vertical plume spread, in meters, a function of atmospheric stability and distance.

Σ_y is lateral plume spread, in meters, with meander and building wake effects (in meters), a function of atmospheric stability, wind speed, and distance [for distances of 800 m or less, $\Sigma_y = M\sigma_y$, where M is determined from Reg. Guide 1.145 Fig. 3; for distances greater than 800 m, $\Sigma_y = (M-1)\sigma_{y\,800\,m} + \sigma_y$.

A is the smallest vertical-plane cross-sectional area of the reactor building, in m^2. (Other structures or a directional consideration may be justified when appropriate.)

Plume meander is only considered during neutral (D) or stable (E, F, or G) atmospheric stability conditions. For such, the higher of the values resulting from Equations 2.3.6-1 and 2.3.6-2 is compared to the value of Equation 2.3.6-3 for meander, and the lower value is selected. For all other conditions (stability classes A, B, or C), meander is not considered and the higher χ/Q value of Equations 2.3.6-1 and 2.3.6-2 is selected.

The χ/Q values calculated at the EAB and LPZ based on meteorological data values representing a 1-hour average are assumed to apply for an entire 2-hour period.

To determine the "maximum sector 0-2 hour χ/Q" value at the EAB, PAVAN constructs a cumulative frequency probability distribution (probabilities of a given χ/Q value being exceeded in that sector during the total time) for each of the 16 sectors using the χ/Q values calculated for each hour of data. This probability is then plotted versus the χ/Q values and a smooth curve is fitted to form an upper bound of the computed points. For each of the 16 curves, the χ/Q value that is exceeded 0.5 percent of the total hours is selected and designated as the sector χ/Q value. The highest of the 16 sector χ/Q values is the maximum sector χ/Q.

The "maximum sector 0-2 hour χ/Q" value at the LPZ is calculated analogously to the EAB. Determination by PAVAN of the LPZ maximum sector χ/Q for periods greater than 0-2 hours is based on a logarithmic interpolation between the 2-hour sector χ/Q and the annual average χ/Q for the same sector. For each time period, the highest of these 16 sector χ/Q values is identified as the maximum sector χ/Q value. The maximum sector χ/Q values will, in most cases, occur in the same sector. If they do not occur in the same sector, all 16 sets of values are used in dose assessment requiring time-integrated concentration considerations. The set that results in the highest time-integrated dose within a sector is considered the maximum sector χ/Q.

The "5% overall site χ/Q" values for the EAB and LPZ are each determined by constructing an overall cumulative probability distribution for all directions. The 0-2 hour χ/Q values computed by PAVAN are plotted versus their probability of being exceeded, and an upper bound curve is fitted by the model. From this curve, the 2-hour χ/Q value that is exceeded 5% of the time is determined. PAVAN then calculates the 5% overall site χ/Q at the LPZ for intermediate time periods by logarithmic interpolation of the maximum of the 16 annual average χ/Q values and the 5% 0-2 hour χ/Q values.

2.3.6.3.1 PAVAN Meteorological Database

The meteorological database utilized for the EAB and LPZ χ/Q calculations were prepared for use in PAVAN by transforming the five years (i.e. 1994-1998) of hourly meteorological tower data observations into a joint wind speed-wind direction-stability class occurrence frequency distribution as shown in Tables 2.3-55 and 2.3-56. In accordance with Regulatory Guide 1.145, atmospheric stability class was determined by vertical temperature difference between the 250 ft and the 30 ft level, and wind direction was distributed into 16- 22.5° sectors.

Seven (7) wind speed categories were defined according to Regulatory Guide 1.23 (Reference 35) with the first category identified as "calm". In the equations shown in Section 2.3.6.3, it should be noted that wind speed appears as a factor in the denominator. This presents an obvious difficulty in making calculations for hours of calm. The minimum wind speed (i.e. based on the wind instrument starting threshold) was set to 0.80 mph, and "calm" winds were assigned a value of 0.4 mph (1/2 the threshold value). The procedure used by PAVAN assigns a direction to each calm hour according to the directional distribution for the lowest non-calm wind-speed class. This procedure is performed separately for the calms in each stability class.

A midpoint was also assumed between each of the Regulatory Guide 1.23 wind speed categories, Nos. 2-6, as to be inclusive of all wind speeds. The wind speed categories have therefore been defined as follows:

CATEGORY NO.	REGULATORY GUIDE 1.23 SPEED INTERVAL (MPH)	PAVAN-ASSUMED SPEED INTERVAL (MPH)
1 (Calm)	0 to < 1	0 to <0.80
2	1 to 3	≥0.80 to <3.5
3	4 to 7	≥3.5 to <7.5
4	8 to 12	≥7.5 to <12.5
5	13 to 18	≥12.5 to <18.5
6	19 to 24	≥18.5 to <24
7	>24	≥24

2.3.6.3.2 PAVAN Model Input Parameters

Both the Unit 1 and Unit 2 Containment Building outer wall and the midpoint between the two reactors do not qualify as "elevated" release locations per Regulatory Guide 1.145; therefore, PAVAN requires that release height be assigned a value of 10 m. For this non-elevated release, EAB and LPZ receptor terrain elevation is assumed to be equal to plant grade.

The Containment Building height of 60.7 m, above grade, and the calculated Containment Building vertical cross-sectional area of 2,916.7 m^2 were used for both the EAB and LPZ PAVAN computations (see Byron Drawing A-4).

2.3.6.3.3 PAVAN EAB and LPZ χ/Q

Atmospheric χ/Q diffusion estimates predicted by PAVAN at the EAB and LPZ are summarized below.

EAB and LPZ χ/Q SUMMARY (sec/m^3)
BYRON STATION

Release Point	Receptor	0-2 hour	0-8 hour	8-24 hour	1-4 day	4-30 day
Unit 1 and Unit 2 Outer Containment Wall	EAB (445 m)	5.36E-04	2.65E-04	1.89E-04	9.04E-05	3.14E-05
Midpoint between the Unit 1 and 2 Reactors	LPZ (4828 m)	3.90E-05	1.67E-05	1.10E-05	4.39E-06	1.18E-06

2.3.6.4 Calculation of χ/Q at the Control Room Intakes

Calculations of atmospheric diffusion factors (χ/Q) at each of the two Control Room Intakes (i.e. Fresh Air and Turbine Building Emergency Air) were performed for releases from the following four points: The Containment Wall, the Plant Vent, the PORVs/Safety Valves, and the Main Steam Line Break (MSLB) for periods of 2, 8, and 16 hours, and for 3 and 26 days. The NRC-sponsored computer code ARCON96 (Reference 36) is utilized in accordance with the procedures in Regulatory Guide (RG) 1.194 (Reference 37).

2.3.6.4.1 ARCON96 Model Analysis

The four releases do not qualify as elevated per RG 1.194, since none are equal to or greater than 2.5 times the height of the adjacent structures; therefore, ARCON96 is executed in ground release mode. The basic model for a ground-level release is as follows:

$$\frac{\chi}{Q} = \frac{1}{\pi \sigma_y \sigma_z U} \exp\left[-0.5\left(\frac{y}{\sigma_y}\right)^2\right] \quad (2.3.6\text{-}4)$$

where:

χ/Q = relative concentration (concentration divided by release rate) [(ci/m^3)/(ci/s)]
σ_y, σ_z = lateral and vertical diffusion coefficients (m)
U = wind speed (m/s)
Y = lateral distance from the horizontal centerline of the plume (m)

This equation assumes that the release is continuous, constant, and of sufficient duration to establish a representative mean concentration. It also assumes that the material being released is reflected by the ground. Diffusion coefficients are typically determined from atmospheric stability and distance from the release point using empirical relationships. A diffusion coefficient parameterization from the NRC PAVAN and XOQDOQ (Reference 38) codes is used for σ_y and σ_z.

The diffusion coefficients have the following general form:

$$\sigma = a\ x^{b} + c$$

where x is the distance from the release point, in meters; and a , b, and c are parameters that are functions of stability class. These parameters are defined for three (3) downwind distance ranges: 0 to 100 m, 100 to 1000 m, and greater than 1000 m. The diffusion coefficient parameter values may be found in the listing of Subroutine NSIGMA1 in Appendix A of NUREG/CR-6331 Rev. 1.

Diffusion coefficient adjustments for wakes and low wind speeds are also incorporated by ARCON96 as follows:

In order to estimate diffusion in building wakes, composite wake diffusion coefficients, Σ_y and Σ_z, replace σ_y and σ_z. The composite wake diffusion coefficients are defined as follows:

$$\Sigma_y = \left(\sigma_y{}^2 + \Delta\sigma_{y1}{}^2 + \Delta\sigma_{y2}{}^2\right)^{1/2} \qquad (2.3.6\text{-}5)$$

$$\Sigma_z = \left(\sigma_z{}^2 + \Delta\sigma_{z1}{}^2 + \Delta\sigma_{z2}{}^2\right)^{1/2} \qquad (2.3.6\text{-}6)$$

The variables σ_y and σ_z are the general diffusion coefficients, $\Delta\sigma_{y1}$ and $\Delta\sigma_{z1}$ are the low wind speed corrections, and $\Delta\sigma_{y2}$ and $\Delta\sigma_{z2}$ are the building wake corrections. These corrections are described and evaluated in Ramsdall and Fosmire (Reference 39). The low wind speed corrections are:

$$\Delta\sigma_{y1}{}^2 = 9.13 \times 10^5 \left[1 - \left(1 + \frac{x}{1000U}\right)\exp\left(\frac{-x}{1000U}\right)\right] \qquad (2.3.6\text{-}7)$$

$$\Delta\sigma_{z1}{}^2 = 6.67 \times 10^2 \left[1 - \left(1 + \frac{x}{100U}\right)\exp\left(\frac{-x}{100U}\right)\right] \qquad (2.3.6\text{-}8)$$

The variable x is the distance from the release point to the receptor, in meters, and U is the wind speed in meters per second. It is appropriate to use the slant range distance for x because these corrections are made only when the release is assumed to be at the ground level and the receptor is assumed to be on the axis of the plume. The diffusion coefficient corrections that account for enhanced diffusion in the wake have a similar form. These corrections are:

$$\Delta\sigma_{y2}^{\ 2} = 5.24\times10^{-2}\,U^2 A\left[1-\left(1+\frac{x}{10\sqrt{A}}\right)\exp\left(\frac{-x}{10\sqrt{A}}\right)\right] \qquad (2.3.6\text{-}9)$$

$$\Delta\sigma_{z2}^{\ 2} = 1.17\times10^{-2}\,U^2 A\left[1-\left(1+\frac{x}{10\sqrt{A}}\right)\exp\left(\frac{-x}{10\sqrt{A}}\right)\right] \qquad (2.3.6\text{-}10)$$

The constant A is the cross-sectional area of the building.

A conservative upper limit placed on Σ_y is the standard deviation associated with a concentration uniformly distributed across a sector with width equal to the circumference of a circle, and with a radius equal to the distance between the source and receptor. This value is

$$\Sigma_{ymax} = \frac{2\pi x}{\sqrt{12}}$$

$$\approx 1.81x \qquad (2.3.6\text{-}11)$$

2.3.6.4.1.1 ARCON96 Meteorological Database

The 1994-1998 meteorological database utilized by ARCON96 consists of hourly meteorological data observations of wind speed and direction measured at 30 and 250 ft, and delta temperature stability class measured between 250 and 30 ft.

The calm wind occurrences, defined to have a value of 0.4 mph (1/2 the wind instrument threshold starting speed), were reset to the ARCON96 default minimum wind speed value of 0.5 meters per second per RG 1.194, Table A-2.

2.3.6.4.1.2 ARCON96 Input Parameters

ARCON96 is executed for each source/receptor combination shown below:

1) Unit 1 Containment Wall to Control Room Fresh Air Intake
2) Unit 1 Containment Wall to Control Room Turbine Building Emergency Air Intake.
3) Unit 1 Plant Vent to Control Room Fresh Air Intake
4) Unit 1 Plant Vent to Control Room Turbine Building Emergency Air Intake
5) Unit 1 PORVs/Safety Valves to Control Room Fresh Air Intake
6) Unit 1 PORVs/Safety Valves to Control Room Turbine Building Emergency Air Intake
7) Unit 1 MSLB to Control Room Fresh Air Intake
8) Unit 1 MSLB to Control Room Turbine Building Emergency Air Intake
9) Unit 2 Containment Wall to Control Room Fresh Air Intake
10) Unit 2 Containment Wall to Control Room Turbine Building Emergency Air Intake
11) Unit 2 Plant Vent to Control Room Fresh Air Intake
12) Unit 2 Plant Vent to Control Room Turbine Building Emergency Air Intake
13) Unit 2 PORVs/Safety Valves to Control Room Fresh Air Intake
14) Unit 2 PORVs/Safety Valves to Control Room Turbine Building Emergency Air Intake
15) Unit 2 MSLB to Control Room Fresh Air Intake
16) Unit 2 MSLB to Control Room Turbine Building Emergency Air Intake

All release scenarios are conservatively assumed to have a zero (0) vertical velocity, exhaust flow and stack radius. Other ARCON96 input parameter values were set in accordance with RG 1.194, Table A-2 (e.g. surface roughness length = 0.2 m; wind direction window = 90 degrees, 45 degrees on either side of line of sight from source to receptor; minimum wind speed = 0.5 m/s; and averaging sector width constant = 4.3).

Each Containment Wall scenario was modeled as a “diffuse area” source in ARCON96. The method of modeling this release as a diffuse area is in conformance with RG 1.194, as described in Appendix A. All other scenarios are modeled as ground-level point sources.

The area source representation in ARCON96 requires the building cross-sectional area to be calculated from the maximum building dimensions projected onto a vertical plane perpendicular to the line of sight from the building to the intake. The Containment Building, with a height of 195 ft (not including the dome portion above the collar) and a width of 161 ft has an area of 31,395 ft^2 (2,916.7 m^2) (see Byron Drawing A-4). The diffuse area source also requires the release height to be assumed at the vertical center of the projected area, and the initial diffusion coefficients to be specified. Per RG 1.194, Section 3.2.4.4, the initial diffusion coefficients are calculated as follows:

$$\sigma_{Y_0} = \frac{Width_{area\ source}}{6} \qquad (2.3.6\text{-}12) \qquad\qquad \sigma_{Z_0} = \frac{Height_{area\ source}}{6} \qquad (2.3.6\text{-}13)$$

$$\sigma_{Y_0} = \frac{161 ft}{6} = 26.83\ ft = 8.18\ m \qquad\qquad \sigma_{Z_0} = \frac{195 ft}{6} = 32.5\ ft = 9.9\ m$$

The remaining three releases (i.e. Plant Vent, PORVs/Safety Valves, and MSLB) are each modeled as a point source. Per RG 1.194 Table A-2, the building area perpendicular to the wind direction should be utilized. For the PORVs/Safety Valves, the Containment Building area of 2,916.7 m^2 was utilized for both stations. There is no change in this building area with a change in wind direction due to its circular shape. The Auxiliary Building area was utilized for the Plant Vent and MSLB scenarios.

ARCON96 requires input of a horizontal source-receptor distance, defined in RG 1.194 Section 3.4 as "the shortest horizontal distance between the release point and the intake". However, for releases in building complexes, a "taut string length" can be utilized as justifiable. For the MSLB to Control Room Fresh Air Intake scenarios, this "taut string length" was utilized to account for the intervening Auxiliary Building. As per NRC interpretation of this RG, when the "taut string length" is utilized, the intake and release heights should be set equal to each other so as not to effectively double-count the advantage of the slant distance that ARCON96 calculates. Therefore, the intake height was set equal to the release height of 7.9 m.

The ARCON96 input parameter data used in calculating the χ/Q values are summarized in Table 2.3-57.

2.3.6.4.1.3 ARCON96 Control Room Intake χ/Q

A summary of the atmospheric diffusion estimates for the two Control Room Intakes is shown in Table 2.3-58.

2.3.7 References

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TABLE 2.3-1

CLIMATOLOGICAL DATA FROM WEATHER STATIONS SURROUNDING THE BYRON SITE*

PARAMETER	STATION ROCKFORD	CHICAGO (MIDWAY)	ARGONNE
Temperature (°F)			
Annual average	48.1	50.6	47.7
Maximum	103 (July 1955)	104 (June 1953)	101 (July 1956)
Minimum	-22 (Feb. 1976)	-16 (Jan. 1966)	-20 (Jan. 1963)
Degree-days	6845	6127	6911
Relative Humidity (%)			
Annual average at:			
6 a.m.	83	76	87
12 noon	61	59	62
Wind			
Annual average speed (mph)	9.9	10.4	7.6**
Prevailing direction	WNW	W	SW
Fastest mile:			
Speed (mph)	54 (Apr. 1953)	60 (Nov. 1972)	64*** (July 1957)
Direction	ESE	SW	****
Precipitation (in.)			
Annual average	36.72	34.44	31.49
Monthly maximum	11.81 (July 1952)	14.17 (Sept. 1961)	13.17 (Sept. 1961)
Monthly minimum	0.01 (Oct. 1952)	0.20 (Oct. 1964)	0.03 (Jan. 1961)
24-hour maximum	5.56 (Sept. 1961)	6.24 (July 1967)	6.54 ****
Snowfall (in.)			
Annual average	33.4	38.3	****
Monthly maximum	22.7 (Mar. 1964)	33.3 (Dec. 1951)	****
24-hour maximum	10.9 (Feb. 1960)	19.8 (Jan. 1967)	****

TABLE 2.3-1 (Cont'd)

PARAMETER	STATION ROCKFORD	CHICAGO (MIDWAY)	ARGONNE
Mean Annual (no. of days)			
Precipitation ≥0.1 in.	114	123	110
Snow, sleet, hail ≥1.0 in.	11	12	****
Thunderstorms	42	110	****
Heavy fog	23	13	****
Maximum temperature ≥90°F	13	21	****
Minimum temperature ≤32°F	53	119	****

*The data presented in this table are based upon References 2, 3, and 4. For the Rockford and Midway data, the period of records used for these statistics range from 13 to 30 years in length within the time period 1941-1976. The period of record for the Argonne data is the 15-year period 1950-1964.

**Wind at 19-foot tower level.

***Peak gust wind at 19-foot level.

****Data not available.

TABLE 2.3-2

MEASURES OF GLAZING IN VARIOUS SEVERE WINTER STORMS

FOR THE STATE OF ILLINOIS

STORM DATE	RADIAL THICKNESS OF ICE ON WIRE (in.)	RATIO OF ICE WEIGHT TO WEIGHT OF 0.25 in. TWIG	WEIGHT OF ICE (oz.) ON 1 FOOT OF STANDARD (#12) WIRE	CITY	STATE SECTION
2-4 Feb. 1883	-	-	11	Springfield	WSW
20 Mar. 1912	0.5	-	-	Decatur	C
21 Feb. 1913	2.0	-	-	La Salle	NE
11-12 Mar. 1923	1.6	-	12	Marengo	NE
17-19 Dec. 1924	1.2	15:1	8	Springfield	WSW
22-23 Jan. 1927	1.1	-	2	Cairo	SE
31 Mar. 1929	0.5	-	-	Moline	NW
7-8 Jan. 1930	1.2	-	-	Carlinville	WSW
1-2 Mar. 1932	0.5	-	-	Galena	NW
7-8 Jan. 1937	1.5	-	-	Quincy	W
31 Dec. 1947 - 1 Jan. 1948	1.0	-	72	Chicago	NE
10 Jan. 1949	0.8	-	-	Macomb	W
8 Dec. 1956	0.5	-	-	Alton	WSW
20-22 Jan. 1959	0.7	12:1	-	Urbana	E
26-27 Jan. 1967	1.7	17:1	40	Urbana	E

Note: Based on Reference 15.

TABLE 2.3-3

WIND-GLAZE THICKNESS RELATIONS FOR FIVE

PERIODS OF GREATEST SPEED AND GREATEST THICKNESS

	FIVE PERIODS WHEN FIVE FASTEST 5-MINUTE SPEEDS WERE REGISTERED		FIVE PERIODS WHEN FIVE GREATEST ICE THICKNESSES WERE MEASURED	
RANK	SPEED (mph)	ICE THICKNESS (in.)	ICE THICKNESS (in.)	SPEED (mph)
1	50	0.19	2.87	30
2	46	0.79	1.71	18
3	45	0.26	1.50	21
4	40	0.30	1.10	28
5	35	0.78	1.00	18

NOTE: From data collected throughout the United States during the period 1926-1937,

Based on Reference 15.

TABLE 2.3-4

WIND ROSE DATA FOR 30-FOOT AND 250-FOOT LEVELS AT BYRON (1974-1976)

(Values in % of Total Observations)

30-FOOT LEVEL

SPEED (MPS)	N	ENE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
CALM																	1.89
0.26 - 1.50	.43	.30	.28	.26	.28	.28	.26	.30	.49	.54	.62	.53	.78	.62	1.19	.72	7.86
1.51 - 3.00	1.19	.97	.90	.89	1.26	1.06	1.21	1.48	2.14	2.05	1.67	1.47	1.40	1.46	1.47	1.31	21.92
3.01 - 7.00	2.80	2.58	2.47	2.87	2.49	1.92	2.33	3.51	5.82	5.09	4.41	3.68	3.65	3.52	3.76	2.93	53.83
GT. - 7.0	.53	.38	.81	.93	.53	.52	.35	.75	1.48	1.72	1.34	1.15	1.29	1.07	.97	.69	14.51
TOTALS	4.96	4.24	4.47	4.97	4.55	3.77	4.18	6.04	9.93	9.41	8.05	6.85	7.14	6.68	7.42	5.66	100.00

250-FOOT LEVEL

SPEED (MPS)	N	ENE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
CALM																	0.18
0.26 - 1.50	.13	.09	.09	.04	.12	.06	.08	.12	.11	.15	.13	.13	.14	.19	.16	.14	1.87
1.51 - 3.00	.45	.32	.37	.34	.35	.28	.30	.37	.42	.58	.51	.51	.47	.54	.51	.53	6.84
3.01 - 7.00	2.40	2.42	2.47	2.02	1.61	1.70	1.78	2.21	3.13	3.49	3.29	3.49	3.30	3.34	3.47	2.70	42.82
GT. - 7.0	1.54	1.61	2.32	2.54	2.18	2.33	2.35	3.31	5.99	6.21	4.29	2.93	3.28	3.14	2.45	1.82	48.29
TOTALS	4.52	4.45	5.25	4.95	4.26	4.37	4.51	6.01	9.64	10.43	8.22	7.07	7.19	7.21	6.60	5.19	100.00

TABLE 2.3-5

PERSISTENCE OF WIND DIRECTION AT BYRON 30-FOOT LEVEL (1974-1976)

(Number of Occurrences)

PERSISTENCE	WIND DIRECTION																
(hr)	CALM	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1-3	148	519	478	520	465	488	451	495	613	817	893	853	799	795	786	760	596
4-6	20	58	43	56	35	44	42	48	90	131	113	89	87	72	85	97	61
7-9	6	14	11	12	26	19	8	10	12	33	36	28	9	25	10	30	25
10-12	2	2	8	5	9	4	4	3	7	23	11	9	4	3	4	6	1
13-15	0	3	0	2	7	1	1	1	0	4	5	2	1	2	1	1	0
16-18	0	0	0	0	2	0	0	0	1	3	1	0	0	2	2	1	0
19-21	0	0	0	1	0	0	0	0	0	0	0	0	0	1	0	0	0
22-24	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
25-27	0	0	0	0	0	0	0	0	0	1***	0	0	0	0	0	0	0
28-30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
31-33	0	0	0	1**	0	0	0	0	0	0	0	0	0	0	0	0	0
34-39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
40-45	0	1*	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

TABLE 2.3-5 (Cont'd)

NOTES:

* Wind direction persistence of 42 hours from the north was recorded at the 30-foot level from July 10, 1974, into July 11, 1974. The onsite wind speed and stability conditions for this period are presented in Table 2.3-5a. The synoptic weather pattern indicates that wind direction should have been generally from the southeast during the period of persistence. At the beginning of the period (July 10, 1974), a polar cold front was located north of the Byron site. Wind directions recorded near Byron were generally from the southeast which is characteristic of the synoptic situation. By the end of the period, the cold front had moved southeast and past Byron Station. A high pressure center was located near Ft. Wayne. Wind directions observed at locations surrounding the Byron site were again generally from the southeast which reflects the synoptic situation.

  The synoptic flow pattern for the period of wind direction persistence does not support the onsite observations. The validity of the onsite wind direction data for this period is subject to question.

** Wind direction persistence of 31 hours from the northeast was recorded at the 30-foot level from March 12, 1974, into March 13, 1974. The onsite wind speed and stability conditions for the period are presented in Table 2.3-5a. The synoptic weather pattern for this period indicates that a strong pressure gradient was induced by a polar high pressure center located over Hudson Bay. The high moved slowly south-southeast during the period. Regional wind directions observed near the Byron site were generally northeast throughout the period. The wind direction persistence is consistent with the synoptic flow pattern.

*** Wind direction persistence of 25 hours from the south was recorded at the 30-foot level from January 4, 1976, into January 5, 1976. The onsite wind speed and stability conditions for the period are presented in the attached table. The synoptic weather pattern for the period indicates a high-pressure center located to the southeast of the Byron site. This high pressure center induced a generally southerly flow pattern during the period of persistence. The wind direction persistence is consistent with the synoptic flow pattern.

TABLE 2.3-5a

ONSITE WIND SPEED AND STABILITY ASSOCIATED WITH WIND DIRECTION PERSISTENCE

	NORTH			NORTHEAST			SOUTH		
HOUR	DATE	WIND SPEED (M/S)	STABILITY	DATE	WIND SPEED (M/S)	STABILITY	DATE	WIND SPEED (M/S)	STABILITY
1	7/10/74	5.36	C	3/12/74	4.92	E	1/4/76	2.68	F
2		5.36	D		8.49	E		1.79	F
3		4.92	E		8.05	E		2.68	F
4		5.81	E		6.70	E	1/5/76	3.13	F
5		5.81	E		7.15	E		2.68	F
6		4.47	E		8.49	E		2.68	G
7		2.68	E		8.49	E		3.58	F
8		3.13	E		8.94	E		4.02	E
9		4.02	E		8.05	E		5.36	E
10		2.68	E		8.94	D		6.26	E
11	7/11/74	3.58	E		9.39	D		6.70	E
12		3.13	E		9.39	D		7.60	E
13		3.58	E		9.39	C		7.15	E
14		4.47	E		8.94	D		8.94	E

TABLE 2.3-5a (Cont'd)

	NORTH			NORTHEAST			SOUTH		
HOUR	DATE	WIND SPEED (M/S)	STABILITY	DATE	WIND SPEED (M/S)	STABILITY	DATE	WIND SPEED (M/S)	STABILITY
15	7/11/74 (Cont'd)	4.92	E	3/12/74 (Cont'd)	8.05	D	1/5/76 (Cont'd)	8.49	E
16		3.58	E		8.05	D		8.05	E
17		3.58	E		7.15	E		7.60	E
18		4.02	E		6.26	E		7.60	E
19		4.02	E		5.36	E		6.26	E
20		4.02	D		6.26	E		6.70	E
21		3.58	D		4.92	E		7.60	E
22		4.02	D		5.81	E		8.05	E
23		3.58	D		5.36	E		7.60	E
24		4.47	D		4.92	E		6.26	E
25		4.47	D	3/13/74	4.92	E		4.92	E
26		5.36	D		5.81	E			
27		5.36	E		6.70	E			
28		5.36	E		5.81	E			
29		4.92	E		5.81	E			

TABLE 2.3-5a (Cont'd)

	NORTH			NORTHEAST			SOUTH		
HOUR	DATE	WIND SPEED (M/S)	STABILITY	DATE	WIND SPEED (M/S)	STABILITY	DATE	WIND SPEED (M/S)	STABILITY
30	7/11/74 (Cont'd)	3.58	E	3/13/74 (Cont'd)	5.36	E			
31		4.02	E		4.47	D			
32		4.02	E						
33		3.13	E						
34		3.13	E						
35	7/12/74	3.13	E						
36		3.13	E						
37		2.68	F						
38		1.79	F						
39		2.23	E						
40		2.68	E						
41		4.02	E						
42		4.02	E						

TABLE 2.3-6

PERSISTENCE OF WIND DIRECTION AT BYRON 250-FOOT LEVEL (1974-1976)

(Number of Occurrences)

PERSISTENCE	WIND DIRECTION																
(HOURS)	CALM	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1-3	25	421	380	423	374	381	392	393	480	589	701	678	664	701	657	551	459
4-6	1	62	54	70	64	55	49	54	72	133	132	106	100	78	98	77	66
7-9	0	12	20	22	14	14	13	20	24	39	44	36	30	28	32	34	25
10-12	0	8	6	7	12	4	6	7	8	15	19	8	5	8	8	9	5
13-15	0	1	1	4	4	1	2	0	6	11	8	4	0	3	4	1	2
16-18	0	1	2	2	0	0	0	0	1	3	6	1	1	3	0	2	1
19-21	0	0	0	1	0	1	0	0	0	2	3	1	0	0	1	3	0
22-24	0	0	0	0	0	0	1	0	1	0	1	1	0	0	0	0	0
25-27	0	0	0	0	1	1	0	0	0	1	0	0	0	0	0	0	0
28-30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
31-33	0	0	0	0	1	0	0	0	0	1	0	0	0	0	0	0	0
34-39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
40-45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

TABLE 2.3-7

PERSISTENCE OF WIND DIRECTION AT ARGONNE (1950-1964)

(Number of Occurrences)

30-FOOT LEVEL
Direction (36 points)

Hours Of Persistence	CALM	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	Missing
	924	1961	1868	1772	1429	1913	1955	1868	1789	1900	1929	1965	1989	1970	1929																							

TABLE 2.3-8

PERSISTENCE AND FREQUENCY OF WIND DIRECTION AT ROCKFORD (1966-1975)

(Number of Occurrences)

PERSISTENCE (HOURS)*	WIND DIRECTION																
	CALM	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
3	704	721	542	508	646	736	595	719	944	1362	1187	914	810	975	917	905	727
6	213	185	158	97	171	227	110	151	182	419	250	188	149	236	280	220	187
9	98	101	52	33	67	95	28	43	59	178	68	55	49	83	101	73	51
12	30	51	25	8	16	42	7	15	26	98	27	15	8	47	29	33	19
15	7	32	6	3	11	18	4	7	9	46	9	6	3	18	21	12	3
18	1	15	3	1	4	2	1	6	1	29	3	1	0	11	13	4	3
21	0	6	2	1	4	7	0	0	0	11	1	1	1	5	7	7	1
24	0	5	1	0	2	4	0	0	1	11	1	0	0	3	4	1	0
27	0	1	1	0	3	0	0	0	0	7	0	0	0	3	2	1	1
30	0	1	1	0	2	0	0	0	0	1	0	0	0	1	2	0	2
33	0	1	0	0	0	0	0	0	0	4	0	0	1	0	0	0	0
36 - 39	0	1	0	0	1	0	0	0	0	2	0	0	0	0	0	0	0

TABLE 2.3-8 (Cont'd)

PERSISTENCE (HOURS)*	WIND DIRECTION																
	CALM	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
42 - 45	0	0	0	0	0	0	0	0	0	2	0	0	0	0	0	0	0
>45	0	0	0	0	0	0	0	0	0	2	0	0	0	0	0	0	0
TOTAL HOURS (In Percent)	5.4	6.8	4.1	3.0	4.9	6.3	3.3	4.4	5.6	13.4	7.1	5.3	4.5	7.3	7.5	6.3	4.8

* Number of occurrences are based on observations made once every 3 hours, and each observation is assumed to persist for 3 hours.

TABLE 2.3-9

A COMPARISON OF SHORT-TERM TEMPERATURE DATA

AT BYRON (1974-1976), ROCKFORD (1973-1975), AND CARROLL COUNTY (1974-1976)

(Values in °F)

MONTH	AVERAGE			MAXIMUM			MINIMUM		
	BYRON	ROCKFORD	CARROLL	BYRON	ROCKFORD	CARROLL	BYRON	ROCKFORD	CARROLL
January	21.3	24.1	20.4*	53.6	55	52.8*	-14.7	-21	-12.0*
February	27.1	25.5	28.2	67.3	49	63.9	-11.1	-13	-13.3
March	34.5	35.8	35.3	72.4	73	76.2	15.8	1	2.0
April	47.6	46.4	48.9	81.9	83	82.5	28.4	9	18.6
May	58.5	57.7	59.4	91.4	95	92.3	32.0	32	32.9
June	67.9	68.4	70.5	92.1	91	91.8	41.0	44	48.7
July	72.8	73.4	75.4	93.2	97	93.4	48.0	48	40.0
August	69.1	71.2	70.4*	92.8	94	91.6*	47.7	50	47.7*
September	58.9	59.9	59.2*	86.5	91	85.1*	32.0	29	32.5*
October	49.8	52.9	51.2	84.6	87	88.0	23.6	24	24.7
November	36.8	39.7	38.0	67.0	71	69.2	-6.9	11	-2.6
December	24.9	27.1	25.2	63.3	63	63.3	-14.3	-8	-13.5
Year	47.3	48.7	48.5	93.2	97	93.4	-14.7	-21	-13.5

*Based on 2 years of data.

TABLE 2.3-10

A COMPARISON OF SHORT-TERM TEMPERATURE DATA AT BYRON (1974-1976)

WITH LONG-TERM TEMPERATURE DATA AT ROCKFORD (1966-1975) AND ARGONNE (1950-1964)

(Values in °F)

MONTH	AVERAGE			MAXIMUM			MINIMUM		
	BYRON	ROCKFORD	ARGONNE	BYRON	ROCKFORD	ARGONNE	BYRON	ROCKFORD	ARGONNE
January	21.3	20.3	21	53.6	60	65	-14.7	-22	-20
February	27.2	24.1	26	67.3	54	67	-11.1	-16	-16
March	34.5	34.9	33	72.4	76	79	1.4	1	-9
April	47.6	46.6	47	81.9	86	84	17.6	9	14
May	58.5	57.0	58	91.4	95	90	32.0	24	27
June	67.9	68.2	68	92.1	98	96	41.0	38	34
July	72.8	72.0	71	93.2	98	101	50.4	43	45
August	69.1	70.0	70	92.8	94	96	47.7	44	41
September	58.9	61.9	63	86.5	92	96	32.0	29	32
October	49.8	52.0	53	84.6	90	89	23.6	21	16
November	36.8	37.9	37	67.0	73	77	-6.9	8	-2
December	24.9	27.0	25	63.3	65	62	-14.3	-18	-18
Year	47.3	47.8	48	93.2	98	101	-14.7	-22	-20

TABLE 2.3-11

AVERAGE DAILY MAXIMUM AND MINIMUM TEMPERATURES

AT ROCKFORD (1966-1975)

(Values in °F)

MONTH	MAXIMUM	MINIMUM	RANGE
January	27.3	11.2	16.1
February	31.3	14.1	17.2
March	43.8	26.4	17.4
April	57.3	37.1	20.2
May	68.6	46.3	22.3
June	79.3	57.9	21.4
July	83.1	62.0	21.1
August	81.5	60.3	21.2
September	73.5	52.2	21.3
October	63.2	42.4	20.8
November	45.8	30.3	15.5
December	33.2	18.7	14.5
Year	57.3	38.2	19.1

TABLE 2.3-12

A COMPARISON OF SHORT-TERM RELATIVE HUMIDITY VALUES

AT BYRON AND CARROLL COUNTY

(Values in %)

MONTH	BYRON YEAR	BYRON AVERAGE	BYRON MAXIMUM	BYRON MINIMUM	CARROLL COUNTY YEAR	CARROLL COUNTY AVERAGE	CARROLL COUNTY MAXIMUM	CARROLL COUNTY MINIMUM
January	1974	87.0	100.0	40.2	1976	77.0	100.0	37.8
February	1974	79.3	100.0	22.8	1976	76.0	100.0	38.5
March	1974	79.0	100.0	24.0	1976	66.0	100.0	21.8
April	1974	69.3	100.0	20.1	1976	56.2	95.2	19.3
May	1975	63.8	92.3	18.3	1976	55.1	100.0	17.3
June	1975	72.3	100.0	26.6	1976	62.3	100.0	17.1
July	1975	72.3	100.0	42.7	1976	62.6	100.0	21.4
August	*	*	*	*	1976	72.0	100.0	26.9
September	1976	73.6	100.0	25.5	1976	61.3	100.0	17.8
October	1976	61.6	100.0	6.5	1976	51.2	92.3	14.2
November	1976	66.2	100.0	18.3	1976	63.2	98.7	15.6
December	1976	71.4	100.0	40.3	1976	77.7	100.0	37.6
Yearly		72.3	100.0	6.5		65.1	100.0	14.2

* Data not presented because of low data recovery rate.

TABLE 2.3-13

LONG-TERM RELATIVE HUMIDITY VALUES AT ROCKFORD (1966-1975)

AND ARGONNE (1950-1964)

(Values in %)

	ARGONNE AVERAGES	ROCKFORD AVERAGES			ROCKFORD EXTREMES	
MONTH	MONTHLY	MONTHLY	DAILY MAXIMUM	DAILY MINIMUM	MAXIMUM	MINIMUM
January	81.8	74.4	84	64	100	16
February	79.9	72.5	83	61	100	17
March	75.6	70.3	85	54	100	15
April	69.5	68.1	85	50	100	15
May	68.7	66.7	85	47	100	21
June	71.5	68.9	87	50	100	22
July	73.8	70.1	89	50	100	25
August	76.9	73.6	92	52	100	28
September	72.7	73.7	92	52	100	22
October	71.1	70.6	87	50	100	19
November	75.3	75.5	87	61	100	24
December	81.9	79.3	88	69	100	29
Yearly	74.9	72.0	87	55	100	15

TABLE 2.3-14

A COMPARISON OF SHORT-TERM DEW-POINT TEMPERATURES

AT BYRON AND CARROLL COUNTY

(Values in °F)

	BYRON				CARROLL COUNTY			
MONTH	YEAR	AVERAGE	MAXIMUM	MINIMUM	YEAR	AVERAGE	MAXIMUM	MINIMUM
January	1974	17.3	39.2	-18.3	1976	15.4	32.0	-2.2
February	1974	18.5	42.8	-3.9	1976	24.7	49.6	0.3
March	1974	28.5	55.4	-9.3	1976	37.0	58.6	8.2
April	1974	39.0	69.8	6.8	1976	33.7	62.4	13.0
May	1975	48.1	68.6	22.6	1976	40.9	62.8	17.8
June	1975	59.9	77.9	35.3	1976	54.3	70.5	28.3
July	1975	62.5	77.6	48.2	1976	59.1	71.4	38.1
August		*	*	*	1976	55.1	64.8	36.9
September	1976	47.2	66.6	28.8	1976	46.4	64.2	15.0
October	1976	32.0	57.6	-3.5	1976	27.3	52.4	9.7
November	1976	18.3	53.4	-16.5	1976	19.1	39.8	-4.0
December	1976	8.4	39.7	-20.0	1976	12.0	34.6	-16.4
Yearly		33.4	77.9	-20.0		35.4	71.4	-16.4

*Data not presented because of low data recovery rate.

TABLE 2.3-15

LONG-TERM DEW-POINT TEMPERATURE AT ROCKFORD (1966-1975)

AND ARGONNE (1950-1964)

(Values in °F)

MONTH	ARGONNE AVERAGES	ROCKFORD AVERAGES			ROCKFORD EXTREMES	
	MONTHLY	MONTHLY	DAILY MAXIMUM	DAILY MINIMUM	MAXIMUM	MINIMUM
January	16.5	13.8	22	7	54	-31
February	19.8	16.5	23	10	52	-22
March	25.3	25.9	31	21	59	-6
April	36.0	36.0	41	31	68	7
May	46.0	44.6	50	40	70	18
June	56.3	56.5	60	52	75	23
July	60.7	60.6	64	57	77	39
August	60.5	60.3	64	57	79	41
September	52.3	52.3	57	48	73	21
October	41.8	41.9	47	37	68	14
November	29.3	30.7	35	26	59	0
December	19.5	21.4	27	16	55	-24
Yearly	38.7	38.5	44	34	79	-31

TABLE 2.3-16

A COMPARISON OF SHORT-TERM PRECIPITATION TOTALS (WATER EQUIVALENT)

AT BYRON (1974-1976) AND ROCKFORD (1974-1976)

(Values in inches)

MONTH	1974 BYRON	1974 ROCKFORD	1975 BYRON	1975 ROCKFORD	1976 BYRON	1976 ROCKFORD
January	*	3.55	1.19*	2.38	0.07**	0.59
February	0.74**	1.65	0.25	2.03	1.32**	1.41
March	2.26	2.03	1.27	2.81	3.82**	4.94
April	2.79	3.41	0.03*	2.39	3.15	3.60
May	7.39	6.98	2.36	3.03	2.37	2.38
June	6.38	6.30	2.77*	4.68	2.27	2.16
July	1.58	1.48	1.70	1.30	0.36**	2.05
August	1.85	2.21	2.57	4.08	1.31	2.04
September	0.46	0.35	1.13	0.91	1.10	1.39
October	2.58**	2.41	0.41	0.98	2.44	1.94
November	1.49**	2.31	3.69*	4.24	0.20	0.38
December	0.93	1.63	0.99*	1.95	0.30	0.37

* Data not available.
** Data not considered reliable due to missing hours of measurement.

TABLE 2.3-17

PRECIPITATION (WATER EQUIVALENT) AVERAGES AND EXTREMES

AT ROCKFORD (1966-1975) AND ARGONNE (1950-1964)

(Values in inches)

MONTH	NORMAL TOTAL		MAXIMUM		MINIMUM	
	ROCKFORD	ARGONNE	ROCKFORD	ARGONNE	ROCKFORD	ARGONNE
January	1.51	1.42	3.55	3.52	0.51	0.03
February	1.24	1.33	2.67	2.24	0.04	0.10
March	2.31	2.19	4.53	3.85	1.00	0.23
April	4.71	3.60	9.92	5.37	1.79	1.82
May	4.57	3.08	6.98	5.55	1.83	0.13
June	6.07	3.73	9.98	7.39	2.37	1.03
July	3.53	4.32	8.39	7.05	1.30	1.29
August	3.31	3.43	9.10	6.26	0.67	1.25
September	4.22	2.81	10.63	13.17	0.35	0.86
October	3.41	2.59	8.32	13.03	0.98	0.24
November	2.39	1.72	4.24	3.53	1.36	0.86
December	2.39	1.26	5.04	2.51	1.20	0.35
Year	39.85	31.49	56.48	43.07	27.80	19.78

TABLE 2.3-18

JOINT FREQUENCY DISTRIBUTION OF WIND DIRECTION AND PRECIPITATION

OCCURRENCE FOR ROCKFORD (1966-1975)

(Frequency of joint occurrence in percent)

	WIND DIRECTION																
MONTH	CALM	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
January	0.2	2.6	1.1	0.8	0.7	0.9	0.8	1.4	1.2	2.1	0.7	0.4	1.1	1.4	0.9	0.9	1.0	18.4
February	0.3	1.6	1.1	1.2	1.8	1.4	0.7	0.6	0.6	0.9	0.3	0.4	0.5	0.8	1.0	1.4	1.0	15.7
March	0.0	1.7	1.3	0.6	1.7	1.4	0.7	1.0	0.9	0.9	0.4	0.4	0.2	0.7	0.9	1.3	0.9	15.0
April	0.2	0.9	0.8	0.7	1.5	1.4	0.9	0.8	0.9	0.9	0.6	0.4	0.2	0.8	0.5	0.4	0.5	12.5
May	0.2	0.5	0.8	0.6	1.3	1.2	0.6	0.5	1.1	0.9	0.6	0.4	0.2	0.6	0.3	0.3	0.4	10.4
June	0.0	0.3	0.6	0.5	0.9	0.5	0.4	0.2	0.5	0.8	0.5	0.4	0.5	0.5	0.4	0.2	0.3	7.7
July	0.2	0.4	0.0	0.2	0.4	0.4	0.3	0.2	0.4	0.9	0.5	0.5	0.2	0.1	0.2	0.3	0.1	5.4
August	0.2	0.4	0.4	0.1	0.1	0.4	0.3	0.4	0.2	0.6	0.5	0.3	0.2	0.2	0.2	0.1	0.1	4.6
September	0.2	0.8	0.5	0.3	0.6	0.7	0.4	0.8	0.7	0.9	0.3	0.6	0.4	0.2	0.3	0.0	0.3	7.8
October	0.1	0.8	0.6	0.2	0.4	0.7	0.3	0.7	1.0	1.5	0.6	0.4	0.4	0.3	0.5	0.5	0.2	9.3
November	0.2	1.7	1.2	0.8	1.1	0.6	0.3	0.8	0.7	1.4	1.0	0.6	0.3	1.3	0.8	1.0	1.1	15.1
December	0.5	2.3	1.2	1.3	1.6	1.1	0.9	1.2	1.5	1.4	1.3	0.9	0.8	1.4	1.7	1.6	0.9	23.5
Year	0.2	1.2	0.8	0.6	1.0	0.9	0.6	0.7	0.8	1.2	0.6	0.5	0.4	0.7	0.7	0.7	0.6	12.1

NOTE: Frequencies of joint occurrences are based on observations made once every 3 hours.

TABLE 2.3-19

MAXIMUM PRECIPITATION (WATER EQUIVALENT) FOR

SPECIFIED TIME INTERVALS AT ARGONNE (1950-1964)

(Values in inches)

	TIME INTERVAL (hours)						
MONTH	1	2	3	6	12	36	48
January	0.44	0.63	0.88	1.16	2.04	2.69	2.69
February	0.32	0.58	0.76	0.95	1.00	1.07	1.07
March	0.52	0.68	0.86	1.15	1.43	2.40	2.40
April	1.18	1.34	1.70	2.50	3.00	3.35	3.35
May	1.12	1.26	1.36	1.56	2.29	3.40	3.43
June	2.20	3.28	4.00	4.22	4.23	4.23	4.25
July	1.40	2.00	2.12	2.76	2.90	3.49	3.49
August	1.92	2.32	2.34	2.40	2.78	2.79	2.79
September	1.04	1.44	1.82	2.39	2.56	4.66	4.92
October	1.40	2.44	2.79	3.63	4.98	8.10	8.62
November	0.42	0.62	0.75	0.97	1.67	1.90	1.95
December	0.36	0.48	0.56	0.64	0.90	1.29	1.33
Year	2.20	3.28	4.00	4.22	4.98	8.10	8.62

TABLE 2.3-20

ICE PELLET AND SNOW PRECIPITATION AT ROCKFORD (1966-1975)

(Values in inches of ice and/or snow)

MONTH	AVERAGE	MONTHLY MAXIMUM	24-HOUR MAXIMUM	AVERAGE NUMBER OF HOURS
January	6.7	12.2	9.3	100
February	7.2	18.6	7.6	91
March	6.1	15.7	4.4	61
April	2.4	6.7	6.7	19
May	0.1	1.0	1.0	1
June	0.0	0.0	0.0	0
July	0.0	0.0	0.0	0
August	0.0	0.0	0.0	0
September	0.0	0.0	0.0	0
October	0.2	2.2	2.2	2
November	2.6	6.4	4.5	45
December	8.6	20.0	7.2	104
Year	31.6	20.0	9.3	422

TABLE 2.3-21

PERSISTENCE AND FREQUENCY OF FOG AT ROCKFORD (1966-1975)

(Number of Occurrences)

PERSISTENCE HOURS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	YR
3	29	27	25	45	47	42	56	61	71	29	32	30	494
6	12	15	16	26	18	19	20	38	49	23	17	27	280
9	11	5	9	12	13	12	13	29	16	16	18	14	168
12	6	2	7	10	7	5	6	8	8	6	8	13	86
15	7	2	5	4	5	1	1	4	1	9	12	11	62
18	3	2	5	2	3	1	1	1	5	6	6	4	39
21	4	5	6	2	1	0	0	1	0	3	3	5	30
24	3	0	3	0	1	0	2	0	0	2	3	4	18
27	2	2	0	1	2	0	0	1	1	1	1	1	12
30	2	1	1	1	0	0	0	0	1	0	1	0	7
33	2	1	1	0	0	0	0	0	1	0	1	2	8

TABLE 2.3-21 (Cont'd)

PERSISTENCE HOURS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	YR
36-39	1	1	0	0	1	0	0	0	0	0	4	4	11
42-45	0	3	3	0	0	0	0	0	0	2	2	1	11
>45	3	1	2	0	0	1	0	0	0	0	1	6	14
ANNUAL AVERAGE TOTAL HOURS OF FOG	105	74	104	71	71	50	56	89	94	91	126	165	1096

NOTE: Number of occurrences are based on observations made once every 3 hours, and each observation is assumed to persist for 3 hours.

TABLE 2.3-22

FOG DISTRIBUTION BY THE HOUR OF THE DAY

AT ROCKFORD (1966-1975)

(Values in %)

HOUR OF THE DAY	WINTER	SPRING	SUMMER	FALL	YEAR
0	12	14	13	12	13
3	14	17	25	17	17
6	14	24	42	27	25
9	16	12	7	16	14
12	12	8	3	8	8
15	11	7	2	6	7
18	11	9	2	6	8
21	11	9	5	9	9

TABLE 2.3-23

FREQUENCY OF PASQUILL STABILITY CLASSES AT BYRON (1974-1976)

(Frequency of Occurrence in Percent of Total Monthly Observations)

MONTH	PASQUILL STABILITY CLASS A	B	C	D	E	F	G
January	0.29	1.25	2.49	45.02	40.18	9.00	1.77
February	0.98	1.25	3.09	42.78	40.88	8.96	2.06
March	0.60	2.09	4.99	38.53	44.79	7.35	1.65
April	0.22	2.50	3.95	31.02	47.92	11.90	2.50
May	2.19	2.86	4.67	36.89	41.18	10.96	1.24
June	8.52	4.48	4.92	30.85	34.20	12.37	4.67
July	13.72	7.76	5.30	18.96	30.93	16.17	7.16
August	9.71	5.60	3.17	19.69	36.02	19.04	6.77
September	6.67	4.95	3.43	21.20	36.39	18.24	9.12
October	2.71	3.98	2.32	21.42	41.91	18.39	9.28
November	0.50	1.23	2.08	36.46	47.45	9.25	3.03
December	1.18	2.13	1.89	38.78	46.42	8.11	1.48
Annual Average	4.06	3.39	3.46	31.41	40.54	12.71	4.43

Data for this table were derived from the three-way joint frequency distribution of wind direction, wind speed, and Pasquill stability class for the period of record.

TABLE 2.3-24

PERSISTENCE OF PASQUILL STABILITY CLASSES AT BYRON (1974-1976)

(Number of Occurrences)

PERSISTENCE (HOURS)	PASQUILL STABILITY CLASS						
	A	B	C	D	E	F	G
1-3	221	508	573	1167	1136	772	196
4-6	65	26	19	290	394	192	69
7-9	34	4	1	142	185	60	33
10-12	4	0	0	63	127	21	11
13-15	0	0	0	41	60	4	1
16-18	1	0	0	15	35	0	0
19-21	0	0	0	16	14	0	0
22-24	0	0	0	6	10	0	0
25-27	0	0	0	6	4	0	0
28-30	0	0	0	4	4	0	0
31-33	0	0	0	4	1	0	0
34-39	0	0	0	6	5	0	0
40-45	0	0	0	3	1	0	0
>45	0	0	0	4	2	0	0

TABLE 2.3-25

THREE-WAY JOINT FREQUENCY DISTRIBUTION OF WIND DIRECTION, WIND SPEED,

AND PASQUILL STABILITY CLASS FOR THE 30-FOOT LEVEL AT BYRON (1974-1976)

(Values in % of Total Observations)

STABILITY CLASS A

SPEED (MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
CALM																	.05
1-3	.01	.02	.00	.00	.01	.00	.02	.01	.00	.00	.00	.02	.01	.00	.02	.03	.19
4-7	.07	.07	.06	.04	.06	.09	.12	.06	.07	.03	.06	.06	.09	.11	.07	.11	1.18
8-12	.08	.03	.10	.06	.04	.05	.07	.11	.10	.19	.16	.09	.18	.15	.14	.11	1.64
13-18	.00	.00	.02	.00	.01	.01	.05	.06	.08	.17	.15	.08	.07	.09	.07	.03	.90
19-24	.00	.00	.00	.00	.00	.00	.00	.02	.00	.01	.00	.01	.01	.02	.01	.00	.10
>24	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.01	.00	.00	.02
TOTALS	.16	.12	.19	.11	.12	.15	.25	.26	.26	.41	.37	.26	.36	.39	.32	.27	4.06

STABILITY CLASS B

SPEED (MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
CALM																	.05
1-3	.01	.01	.00	.00	.01	.00	.01	.00	.01	.01	.01	.01	.03	.01	.02	.00	.16
4-7	.06	.05	.05	.02	.03	.04	.08	.06	.05	.11	.11	.07	.06	.11	.09	.08	1.05
8-12	.07	.05	.04	.03	.05	.01	.08	.11	.11	.20	.09	.09	.10	.08	.08	.09	1.27
13-18	.02	.03	.01	.02	.04	.03	.04	.04	.04	.10	.09	.06	.08	.06	.08	.02	.75
19-24	.00	.00	.00	.00	.00	.00	.02	.00	.01	.00	.00	.04	.01	.01	.00	.01	.13
> 24	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.01
TOTALS	.16	.13	.11	.08	.12	.09	.23	.21	.22	.42	.29	.27	.27	.28	.27	.21	3.39

TABLE 2.3-25 (Cont'd)

STABILITY CLASS C

SPEED (MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
CALM																	.02
1-3	.01	.01	.00	.00	.02	.00	.01	.00	.02	.00	.01	.00	.01	.02	.01	.03	.19
4-7	.03	.03	.01	.03	.06	.05	.06	.04	.06	.08	.09	.08	.08	.08	.08	.03	.88
8-12	.06	.02	.03	.03	.04	.02	.08	.06	.08	.09	.14	.06	.08	.09	.11	.10	1.09
13-18	.04	.03	.05	.06	.03	.05	.08	.06	.03	.07	.07	.04	.13	.10	.10	.07	1.00
19-24	.00	.00	.01	.00	.01	.01	.01	.02	.01	.03	.01	.03	.03	.01	.02	.01	.23
> 24	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.01	.01	.01	.00	.05
TOTALS	.13	.09	.11	.14	.15	.13	.25	.18	.20	.28	.32	.22	.35	.31	.33	.24	3.46

STABILITY CLASS D

SPEED (MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
CALM																	.25
1-3	.06	.04	.07	.06	.06	.08	.08	.07	.08	.06	.12	.11	.10	.07	.15	.07	1.29
4-7	.37	.37	.32	.34	.35	.30	.37	.40	.45	.50	.48	.40	.41	.47	.39	.42	6.35
8-12	.55	.58	.44	.50	.49	.38	.55	.65	.81	.84	.84	.78	.97	1.09	1.28	.87	11.60
13-18	.44	.31	.40	.53	.40	.28	.36	.39	.69	.60	.67	.48	.79	.87	.94	.67	8.81
19-24	.05	.08	.19	.12	.05	.17	.06	.13	.13	.22	.16	.21	.38	.23	.23	.14	2.56
> 24	.00	.01	.01	.01	.00	.02	.00	.01	.00	.05	.07	.07	.14	.12	.01	.00	.55
TOTALS	1.47	1.39	1.43	1.56	1.35	1.23	1.42	1.66	2.16	2.27	2.35	2.05	2.79	2.86	3.00	2.17	31.41

TABLE 2.3-25 (Cont'd)

STABILITY CLASS E

SPEED (MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
CALM																	.47
1-3	.13	.12	.12	.07	.12	.12	.09	.07	.13	.21	.18	.18	.26	.21	.36	.21	2.57
4-7	.83	.66	.60	.59	.64	.48	.53	.70	1.06	.96	1.00	1.06	.92	.88	.94	.83	12.68
8-12	.90	.68	.68	.71	.54	.39	.30	.86	1.62	1.46	1.27	1.01	.85	.73	.76	.66	13.42
13-18	.46	.28	.34	.59	.38	.23	.20	.43	1.14	1.21	.87	.57	.43	.37	.34	.29	8.14
19-24	.11	.05	.17	.21	.08	.07	.04	.17	.54	.53	.32	.15	.12	.05	.09	.07	2.76
> 24	.00	.00	.07	.06	.00	.00	.00	.00	.08	.10	.04	.07	.06	.00	.00	.00	.50
TOTALS	2.43	1.79	1.98	2.24	1.76	1.29	1.16	2.23	4.57	4.46	3.68	3.06	2.64	2.25	2.48	2.06	40.54

STABILITY CLASS F

SPEED (MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
CALM																	.47
1-3	.16	.10	.04	.06	.05	.06	.06	.11	.13	.19	.22	.14	.21	.22	.35	.24	2.33
4-7	.25	.18	.18	.23	.39	.37	.41	.72	1.08	.71	.37	.24	.27	.30	.25	.25	6.18
8-12	.09	.06	.08	.18	.21	.15	.25	.51	.98	.32	.10	.14	.05	.04	.03	.07	3.27
13-18	.01	.00	.00	.03	.00	.03	.01	.05	.14	.08	.02	.06	.00	.00	.00	.00	.45
19-24	.00	.00	.00	.00	.00	.00	.00	.00	.01	.00	.00	.00	.00	.00	.00	.00	.02
> 24	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
TOTALS	.51	.35	.30	.50	.64	.61	.73	1.39	2.35	1.29	.70	.58	.54	.56	.62	.57	12.71

TABLE 2.3-25 (Cont'd)

STABILITY CLASS G

SPEED (MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
CALM																	.47
1-3	.05	.03	.05	.07	.03	.03	.04	.06	.12	.08	.06	.07	.18	.10	.35	.20	1.50
4-7	.06	.02	.03	.05	.14	.14	.17	.33	.51	.36	.08	.03	.04	.02	.03	.04	2.06
8-12	.01	.00	.00	.01	.02	.04	.05	.10	.14	.00	.00	.00	.00	.00	.01	.00	.40
13-18	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.01
19-24	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
> 24	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
TOTALS	.12	.05	.08	.13	.19	.21	.26	.50	.78	.44	.13	.10	.22	.12	.39	.25	4.43

Note: The calm category represents conditions with wind speeds less than 0.8 mph, which is the threshold speed for the wind speed and wind direction sensors.

TABLE 2.3-26

THREE-WAY JOINT FREQUENCY DISTRIBUTION OF WIND DIRECTION, WIND SPEED, AND PASQUILL STABILITY

CLASS FOR THE 250-FOOT LEVEL AT BYRON (1974-1976)

(Values in % of Total Observations)

STABILITY CLASS A

SPEED (MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
CALM																	.03
1-3	.00	.00	.00	.00	.01	.00	.01	.01	.00	.00	.01	.01	.00	.01	.01	.00	.09
4-7	.02	.07	.05	.04	.04	.02	.04	.04	.01	.04	.04	.04	.06	.07	.10	.07	.76
8-12	.04	.07	.09	.05	.07	.10	.05	.08	.10	.09	.09	.09	.01	.16	.12	.08	1.38
13-18	.02	.03	.05	.04	.02	.05	.05	.10	.16	.12	.10	.07	.16	.15	.06	.06	1.26
19-24	.00	.00	.00	.02	.00	.01	.07	.10	.09	.12	.10	.08	.04	.06	.08	.00	.79
> 24	.00	.00	.00	.01	.00	.00	.00	.03	.00	.01	.02	.02	.02	.03	.00	.00	.17
TOTALS	.09	.18	.19	.16	.15	.19	.22	.37	.37	.39	.37	.31	.38	.48	.38	.22	4.47

STABILITY CLASS B

SPEED (MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
CALM																	.00
1-3	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.01	.01	.00	.00	.05
4-7	.02	.02	.04	.03	.01	.03	.02	.04	.04	.03	.07	.04	.09	.07	.07	.05	.67
8-12	.02	.04	.05	.03	.03	.06	.06	.12	.14	.14	.09	.06	.08	.05	.08	.08	1.13
13-18	.05	.02	.04	.01	.02	.03	.08	.05	.11	.13	.05	.08	.08	.13	.10	.05	1.03
19-24	.00	.00	.00	.01	.06	.02	.03	.04	.06	.10	.07	.03	.06	.04	.03	.01	.57
> 24	.00	.00	.00	.00	.00	.00	.02	.01	.01	.00	.01	.01	.02	.01	.00	.00	.11
TOTALS	.10	.09	.13	.09	.13	.15	.21	.26	.36	.40	.28	.23	.34	.32	.28	.20	3.58

TABLE 2.3-26 (Cont'd)

STABILITY CLASS C

SPEED (MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
CALM																	.01
1-3	.00	.00	.00	.00	.00	.01	.00	.01	.00	.01	.00	.00	.00	.02	.00	.00	.09
4-7	.02	.01	.01	.02	.05	.02	.04	.04	.03	.03	.07	.07	.05	.07	.03	.05	.62
8-12	.04	.02	.04	.03	.03	.08	.05	.06	.08	.09	.11	.05	.10	.05	.05	.05	.93
13-18	.02	.02	.06	.06	.01	.06	.03	.05	.07	.08	.06	.07	.11	.10	.12	.05	.97
19-24	.00	.00	.03	.04	.00	.06	.03	.06	.05	.06	.06	.04	.07	.09	.05	.03	.68
> 24	.00	.00	.01	.00	.02	.02	.04	.04	.02	.01	.01	.02	.04	.01	.01	.00	.27
TOTALS	.09	.06	.16	.15	.11	.25	.19	.27	.25	.28	.31	.24	.38	.36	.26	.19	3.57

STABILITY CLASS D

SPEED (MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
CALM																	.03
1-3	.05	.04	.04	.01	.05	.02	.03	.06	.07	.05	.06	.05	.08	.07	.05	.05	.79
4-7	.17	.10	.14	.16	.16	.13	.15	.24	.20	.25	.27	.29	.24	.22	.23	.25	3.20
8-12	.34	.38	.34	.37	.24	.33	.39	.52	.48	.54	.53	.54	.50	.54	.60	.45	7.07
13-18	.37	.42	.47	.47	.38	.41	.47	.51	.76	.77	.60	.62	1.06	1.07	.95	.72	10.06
19-24	.20	.22	.37	.40	.24	.24	.29	.42	.61	.53	.51	.47	.81	.64	.49	.49	6.94
> 24	.06	.17	.13	.06	.17	.21	.13	.32	.33	.41	.22	.30	.35	.19	.16	.09	3.30
TOTALS	1.19	1.32	1.48	1.48	1.23	1.34	1.47	2.08	2.46	2.54	2.18	2.27	3.04	2.74	2.48	2.05	31.39

TABLE 2.3-26 (Cont'd)

STABILITY CLASS E

SPEED (MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
CALM																	.05
1-3	.06	.04	.02	.01	.05	.01	.03	.01	.01	.04	.03	.03	.06	.07	.06	.04	.58
4-7	.31	.15	.16	.18	.13	.15	.10	.16	.21	.23	.22	.25	.19	.24	.28	.20	3.18
8-12	.48	.57	.55	.41	.28	.23	.28	.35	.53	.75	.66	.72	.62	.57	.64	.56	8.20
13-18	.85	.69	.86	.74	.54	.37	.38	.56	.98	1.46	1.23	1.10	1.07	1.06	.94	.75	13.58
19-24	.38	.17	.48	.50	.43	.40	.34	.67	1.40	1.71	.96	.48	.47	.45	.31	.26	9.42
> 24	.08	.08	.33	.25	.16	.27	.37	.50	1.04	.81	.29	.23	.08	.06	.03	.07	4.64
TOTALS	2.16	1.69	2.41	2.09	1.59	1.43	1.51	2.26	4.17	5.00	3.40	2.82	2.49	2.45	2.27	1.86	39.64

STABILITY CLASS F

SPEED (MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
CALM																	.00
1-3	.01	.01	.02	.00	.01	.02	.00	.01	.03	.01	.02	.02	.01	.02	.01	.00	.24
4-7	.05	.06	.05	.03	.05	.03	.06	.08	.10	.10	.05	.06	.09	.07	.05	.04	.98
8-12	.13	.15	.17	.10	.10	.09	.09	.09	.26	.17	.15	.20	.15	.26	.27	.20	2.59
13-18	.27	.19	.20	.22	.28	.28	.31	.29	.58	.47	.57	.48	.21	.34	.36	.18	5.23
19-24	.03	.03	.06	.14	.19	.21	.24	.38	.88	.54	.16	.09	.01	.02	.06	.05	3.10
> 24	.00	.00	.00	.00	.01	.05	.03	.11	.28	.11	.00	.00	.00	.00	.00	.00	.60
TOTALS	.49	.43	.50	.50	.66	.67	.74	.96	2.13	1.41	.95	.87	.48	.72	.74	.47	12.74

TABLE 2.3-26 (Cont'd)

STABILITY CLASS G

SPEED (MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
CALM																	.00
1-3	.01	.00	.01	.00	.00	.00	.00	.00	.00	.03	.00	.02	.00	.00	.02	.03	.14
4-7	.09	.03	.05	.03	.06	.03	.02	.02	.01	.06	.03	.04	.03	.03	.04	.05	.62
8-12	.15	.04	.06	.04	.02	.07	.07	.06	.12	.09	.06	.05	.08	.10	.11	.13	1.25
13-18	.07	.04	.05	.01	.09	.20	.14	.12	.23	.19	.20	.14	.07	.09	.07	.09	1.80
19-24	.01	.01	.00	.03	.01	.08	.07	.06	.15	.16	.12	.00	.00	.01	.01	.01	.74
> 24	.00	.00	.00	.00	.00	.02	.00	.00	.01	.00	.00	.00	.00	.00	.00	.00	.05
TOTALS	.33	.13	.18	.12	.19	.41	.29	.26	.52	.52	.41	.26	.18	.24	.26	.30	4.61

Note: The calm category represents conditions with wind speeds less than 0.8 mph, which is the threshold speed for the wind speed and wind direction sensors.

TABLE 2.3-27

THREE-WAY JOINT FREQUENCY DISTRIBUTION

OF WIND DIRECTION, WIND SPEED, AND PASQUILL STABILITY CLASS

FOR ROCKFORD (1966-1975)

(Values in % of Total Observations)

	WIND SPEED (METER/SEC)	CALM	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
										WIND DIRECTION									
	CALM	.13																	.13
	LT. 2.0		.01	.02	.00	.01	.01	.00	.01	.01	.01	.01	.00	.01	.01	.00	.01	.01	.12
·A·	2.0 - 6.0		.00	.00	.00	.01	.00	.00	.00	.01	.01	.00	.00	.01	.01	.00	.00	.01	.08
	GT. 6.0		.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
	TOTAL	.13	.01	.02	.01	.02	.01	.00	.01	.01	.02	.01	.00	.02	.02	.00	.01	.02	.34
	CALM	.68																	.68
	LT. 2.0		.09	.03	.05	.07	.05	.06	.05	.06	.19	.08	.04	.06	.09	.07	.06	.07	1.12
·B·	2.0 - 6.0		.22	.09	.07	.14	.12	.09	.09	.12	.17	.18	.17	.15	.17	.19	.13	.13	2.25
	GT. 6.0		.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
	TOTAL	.68	.31	.12	.13	.21	.17	.15	.14	.18	.36	.27	.21	.21	.25	.26	.20	.20	4.05
	CALM	.51																	.51
	LT. 2.0		.10	.04	.03	.06	.08	.06	.07	.13	.24	.14	.09	.05	.07	.07	.10	.03	1.36
·C·	2.0 - 6.0		.57	.28	.23	.34	.33	.21	.30	.28	.79	.63	.55	.54	.53	.52	.45	.41	6.95
	GT. 6.0		.02	.02	.00	.02	.03	.00	.00	.01	.07	.06	.08	.06	.09	.04	.04	.01	.55
	TOTAL	.51	.69	.34	.27	.42	.43	.27	.37	.42	1.10	.83	.72	.66	.68	.63	.59	.45	9.37
	CALM	.72																	.72
	LT. 2.0		.16	.09	.10	.20	.19	.13	.18	.27	.43	.16	.14	.15	.18	.13	.16	.12	2.79
·D·	2.0 - 6.0		2.96	1.95	1.33	2.10	2.96	1.46	1.95	2.26	4.63	2.51	1.92	1.66	2.46	2.66	2.40	2.03	37.24
	GT. 6.0		1.49	.93	.68	1.11	1.11	.48	.72	.99	2.55	1.64	1.32	1.10	2.48	2.67	1.67	.98	21.93
	TOTAL	.72	4.62	2.98	2.11	3.41	4.26	2.07	2.85	3.52	7.61	4.31	3.38	2.90	5.12	5.46	4.23	3.13	62.68

TABLE 2.3-27 (Cont'd)

WIND SPEED (METER/SEC)		CALM	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
	CALM	.00																	.00
	LT. 2.0		.04	.02	.02	.07	.09	.07	.05	.09	.31	.12	.07	.05	.03	.05	.03	.03	1.13
·E·	2.0 - 6.0		.61	.37	.20	.39	.61	.30	.47	.49	1.31	.67	.37	.34	.70	.67	.73	.47	8.72
	GT. 6.0		.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
	TOTAL	.00	.66	.39	.22	.46	.70	.37	.52	.58	1.63	.79	.44	.39	.73	.72	.76	.50	9.85
	CALM	.99																	.99
	LT. 2.0		.11	.09	.08	.12	.21	.14	.13	.31	.95	.27	.19	.08	.17	.12	.17	.14	3.28
·F·	2.0 - 6.0		.30	.16	.10	.20	.29	.15	.25	.38	1.17	.43	.31	.20	.23	.25	.33	.31	5.05
	GT. 6.0		.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
	TOTAL	.99	.41	.25	.17	.32	.51	.29	.38	.70	2.12	.70	.50	.27	.40	.37	.50	.46	9.32
	CALM	2.40																	2.40
	LT. 2.0		.05	.03	.04	.08	.17	.10	.11	.23	.55	.21	.09	.07	.10	.08	.04	.04	1.99
·G·	2.0 - 6.0		.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
	GT. 6.0		.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
	TOTAL	2.40	.05	.03	.04	.08	.17	.10	.11	.23	.55	.21	.09	.07	.10	.08	.04	.04	4.40
	CALM	5.43																	5.43
	LT. 2.0		.57	.31	.33	.60	.80	.56	.61	1.11	2.70	.98	.61	.47	.64	.52	.57	.43	11.80
·ALL·	2.0 - 6.0		4.67	2.85	1.93	3.18	4.31	2.22	3.06	3.54	8.07	4.43	3.33	2.89	4.10	4.30	4.04	3.37	60.29
	GT. 6.0		1.51	.95	.68	1.13	1.14	.48	.72	.99	2.62	1.70	1.40	1.16	2.57	2.71	1.71	.99	22.48
	TOTAL	5.43	6.75	4.12	2.95	4.91	6.25	3.26	4.39	5.64	13.39	7.11	5.33	4.52	7.31	7.53	6.32	4.79	100.00

TABLE 2.3-28

PERSISTENCE AND FREQUENCY OF PASQUILL STABILITY CLASSES AT ROCKFORD (1966-1975)

(Number of Occurrences)

PERSISTENCE (HOURS)	PASQUILL STABILITY CLASS A	B	C	D	E	F	G
3	84	553	1363	1163	1439	1248	510
6	7	188	414	648	403	446	229
9	0	55	143	278	155	158	92
12	0	22	28	263	38	32	10
15	0	0	1	180	3	4	0
18	0	0	0	138	0	0	0
21	0	0	0	106	0	0	0
24	0	0	0	54	0	0	0
27	0	0	0	49	0	0	0
30	0	0	0	55	0	0	0
33	0	0	0	37	0	0	0
36-39	0	0	0	103	0	0	0

TABLE 2.3-28 (Cont'd)

PERSISTENCE (HOURS)	PASQUILL STABILITY CLASS A	B	C	D	E	F	G
42-45	0	0	0	85	0	0	0
>45	0	0	0	274	0	0	0
Total Hours (in percent)	0.3	4.0	9.4	65.4	9.9	6.6	4.4

NOTE: Number of occurrences are based on observations made once every 3 hours, and each observation is assumed to persist for 3 hours.

TABLE 2.3-28a

FREQUENCY DISTRIBUTION OF VISIBLE PLUME LENGTH

TWO NATURAL DRAFT TOWERS - FULL LOAD

PLUME LENGTH: DISTANCE FROM TOWERS (meters)	SUMMER	WINTER	ANNUAL
100	82.4*	100.0	88.5
200	60.1	100.0	74.7
300	47.1	99.8	65.5
400	39.9	99.2	59.5
500	34.3	98.9	55.0
700	27.6	97.6	48.1
1,000	22.5	95.1	42.4
1,500	18.0	90.1	36.7
2,000	16.0	82.4	32.9
3,000	13.7	72.0	27.8
4,000	12.2	61.5	24.1
5,000	11.1	52.6	20.7
10,000	9.0	29.9	12.6
20,000	7.5	21.5	9.3
30,000	7.0	19.2	8.5

* These columns indicate percentage of time a given plume length is exceeded.

TABLE 2.3-28b

FREQUENCY DISTRIBUTION OF VISIBLE PLUME HEIGHT

TWO NATURAL DRAFT TOWERS - FULL LOAD

HEIGHT (meters)	SUMMER	SPRING-FALL	WINTER	ANNUAL
200	100.0*	100.0	100.0	100.0
500	35.0	24.8	48.2	33.2
750	17.4	9.6	24.7	15.3
1,000	10.7	4.4	15.0	8.6
2,000	4.0	0.8	4.1	2.4
3,000	1.7	0.15	1.2	0.8

* These columns indicate the percentage exceeding a given height.

TABLE 2.3-29

CUMULATIVE FREQUENCY DISTRIBUTION OF X/Q* FOR A 1-HOUR TIME PERIOD

AT THE MINIMUM EXCLUSION AREA BOUNDARY DISTANCE (445 M), BYRON SITE

CHI/Q RANGE			DOWNWIND SECTOR																
GT		LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
5.81-04	to		201	147	92	84	140	96	225	145	78	57	46	50	78	68	71	121	1699
			8.7	7.1	5.4	5.8	8.8	6.5	13.5	11.3	7.1	6.5	5.0	4.8	8.0	8.3	7.6	8.7	7.9
5.23-04	to	5.81-04	11	0	0	0	0	0	2	0	1	0	0	1	4	5	5	9	38
			9.2	7.1	5.4	5.8	8.8	6.5	13.6	11.3	7.2	6.5	5	4.9	8.5	8.9	8.1	9.3	8.0
4.70-04	to	5.23-04	15	16	6	11	14	16	28	19	10	5	5	4	8	7	8	18	190
			9.8	7.9	5.7	6.6	9.7	7.6	15.3	12.8	8.1	7.1	5.5	5.3	9.3	9.8	9.0	10.6	8.9
4.23-04	to	4.70-04	52	41	23	22	22	28	24	18	19	12	16	6	14	16	21	22	356
			12.1	9.8	7.1	8.1	11.1	9.4	16.8	14.2	9.9	8.5	7.3	5.9	10.7	11.7	11.2	12.2	10.6
3.81-04	to	4.23-04	1	0	0	0	0	0	1	1	1	0	0	0	0	0	1	0	5
			12.1	9.8	7.1	8.1	11.1	9.4	16.8	14.3	9.9	8.5	7.3	5.9	10.7	11.7	11.3	12.2	10.6
3.43-04	to	3.81-04	51	58	33	16	26	27	24	21	21	13	9	18	26	22	27	39	431
			14.3	12.6	9.0	9.3	12.7	11.3	18.3	15.9	11.9	10.0	8.2	7.6	13.4	14.4	14.2	14.9	12.6
3.09-04	to	3.43-04	23	27	29	25	40	29	44	24	17	17	19	12	16	18	13	8	361
			15.3	13.9	10.7	11.0	15.2	13.2	20.9	17.8	13.4	11.9	10.3	8.8	15.1	16.6	15.6	15.5	14.3
2.78-04	to	3.09-04	66	32	16	17	9	9	12	12	17	5	8	14	24	30	22	46	339
			18.2	15.5	11.7	12.2	15.8	13.8	21.6	18.7	15.0	12.5	11.2	10.1	17.5	20.3	17.9	18.8	15.8
2.50-04	to	2.78-04	77	27	15	4	8	6	3	9	6	8	7	16	23	21	23	54	307
			21.5	16.8	12.6	12.5	16.3	14.2	21.8	19.4	15.5	13.4	11.9	11.7	19.9	22.9	20.4	22.7	17.2
2.25-04	to	2.50-04	116	69	74	75	75	54	79	61	40	38	40	36	59	42	64	83	1005
			26.5	20.1	16.9	17.7	21.0	17.9	26.5	24.1	19.2	17.8	16.3	15.1	26.0	28.0	27.2	28.6	21.9
2.02-04	to	2.25-04	46	21	7	10	1	4	2	4	11	3	5	9	15	10	10	24	182
			28.5	21.1	17.3	18.4	21.1	18.1	26.7	24.5	20.2	18.1	16.8	16.0	27.6	29.2	28.3	30.3	22.7
1.82-04	to	2.02-04	93	69	36	65	51	50	58	57	52	39	36	35	41	28	39	62	811
			32.5	24.4	19.4	22.9	24.3	21.5	30.2	28.9	24.9	22.6	20.7	19.4	31.8	32.6	32.5	34.8	26.5
1.64-04	to	1.82-04	115	75	86	72	67	76	68	68	62	51	51	61	64	51	49	50	1066
			37.5	28.0	24.5	27.9	28.5	26.6	34.2	34.2	30.6	28.5	26.2	25.3	38.4	38.9	37.7	38.3	31.4
1.48-04	to	1.64-04	25	9	5	4	0	1	0	0	1	1	2	6	3	1	5	12	75
			38.5	28.5	24.8	28.2	28.5	26.7	34.2	34.2	30.7	28.6	26.4	25.8	38.7	39.0	38.2	39.7	31.8

TABLE 2.3-29 (Cont'd)

CUMULATIVE FREQUENCY DISTRIBUTION OF X/Q* FOR A 1-HOUR TIME PERIOD

AT THE MINIMUM EXCLUSION AREA BOUNDARY DISTANCE (445 M), BYRON SITE

CHI/Q RANGE			DOWNWIND SECTOR																
GT		LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
1.33-04	to	1.48-04	116	87	93	86	62	73	63	49	73	64	50	62	55	46	47	67	1093
			43.6	32.7	30.2	34.2	32.4	31.6	38.0	38.0	37.3	35.9	31.9	31.8	44.4	44.6	43.3	44.0	36.8
1.20-04	to	1.33-04	89	60	70	64	46	34	35	37	40	37	42	41	38	21	24	52	730
			47.4	35.6	34.3	38.6	35.3	33.9	40.1	40.9	41.0	40.2	36.4	35.8	48.3	47.2	45.8	47.7	40.2
1.08-04	to	1.20-04	110	93	90	65	73	74	72	63	72	54	47	54	61	41	33	70	1072
			52.2	40.0	39.6	43.2	39.8	38.9	44.4	45.8	47.5	46.4	41.5	41.0	54.6	52.2	49.4	52.7	45.2
9.68-05	to	1.08-04	114	86	81	75	61	51	53	59	66	69	51	54	29	34	42	70	995
			57.1	44.2	44.3	48.4	43.7	42.3	47.6	50.4	53.6	54.3	47.0	46.2	57.6	56.4	53.8	57.7	49.8
8.72-05	to	9.68-05	91	122	101	83	65	76	76	67	64	47	48	39	35	30	42	80	1066
			61.0	50.1	50.3	54.2	47.8	47.5	52.2	55.6	59.4	59.7	52.2	50.0	61.2	60.0	58.3	63.4	54.7
7.84-05	to	8.72-05	61	69	39	41	42	32	23	17	31	20	26	30	13	13	4	31	492
			63.7	53.4	52.6	57.0	50.4	49.6	53.6	56.9	62.2	62.0	55.0	52.8	62.5	61.6	58.8	65.7	57.0
7.06-05	to	7.84-05	91	92	79	57	66	66	63	48	49	48	39	48	30	28	28	51	883
			67.6	57.8	57.2	61.0	54.6	54.1	57.4	60.7	66.7	67.5	59.3	57.5	65.6	65.0	61.8	69.3	61.1
6.35-05	to	7.06-05	139	150	116	94	95	76	95	72	60	56	51	97	63	37	48	64	1313
			73.6	65.1	64.0	67.5	60.5	59.2	63.1	66.3	72.2	73.9	64.8	66.8	72.1	69.6	66.9	73.9	67.2
5.72-05	to	6.35-05	131	139	119	100	109	138	153	88	74	58	50	70	71	53	60	75	1488
			79.3	71.8	71.0	74.5	67.4	68.5	72.3	73.1	78.9	80.6	70.2	73.6	79.5	76.0	73.3	79.3	74.1
5.15-05	to	5.72-05	98	103	72	38	45	40	80	45	40	28	31	44	37	32	34	35	802
			83.5	76.7	75.2	77.2	70.2	71.2	77.1	76.6	82.6	83.8	73.6	77.8	83.3	80.0	76.9	81.8	77.8
4.63-05	to	5.15-05	111	100	82	57	87	84	85	67	52	32	46	66	56	34	46	46	1051
			88.3	81.5	80.0	81.1	75.7	76.9	82.2	81.9	87.3	87.5	78.5	84.2	89.1	84.1	81.8	85.1	82.6
4.17-05	to	4.63-05	52	48	53	27	34	47	36	30	24	16	30	24	12	8	13	39	493
			90.5	83.9	83.1	83.0	77.8	80.0	84.3	84.2	89.5	89.3	81.8	86.5	90.3	85.1	83.2	87.8	84.9
3.75-05	to	4.17-05	67	66	65	37	61	49	47	47	31	11	44	43	21	18	16	22	645
			93.4	87.0	86.9	85.6	81.6	83.3	87.1	87.9	92.3	90.6	86.6	90.6	92.5	87.3	84.9	89.4	87.9
3.38-05	to	3.75-05	25	25	34	30	43	28	22	19	14	11	24	18	5	17	12	20	347
			94.5	88.2	88.9	87.7	84.3	85.2	88.5	89.3	93.6	91.8	89.2	92.4	93.0	89.4	86.2	90.8	89.5
0.00	to	3.38-05	127	244	189	177	249	219	192	137	70	71	100	79	68	87	129	128	2266
			100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

*X/Q values, expressed in (sec/m^3), are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.
Key: 1.33-04 = 1.33 x 10^{-4}.

TABLE 2.3-30

CUMULATIVE FREQUENCY DISTRIBUTION OF X/Q* FOR A 2-HOUR TIME PERIOD

AT THE MINIMUM EXCLUSION AREA BOUNDARY DISTANCE (445 M), BYRON SITE

CHI/Q RANGE			DOWNWIND SECTOR																
GT		LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
5.81-04	to		166	102	59	79	142	87	230	141	69	50	42	53	67	49	50	85	1471
			5.2	3.4	2.3	3.6	6.1	3.9	9.6	7.6	4.3	3.8	3.2	3.6	4.7	4.0	3.5	4.2	4.7
5.23-04	to	5.81-04	34	29	11	7	6	8	14	13	12	2	7	4	12	16	24	26	225
			6.2	4.4	2.8	3.9	6.3	4.3	10.2	8.2	5.1	4.0	3.7	3.9	5.6	5.3	5.2	5.5	5.4
4.70-04	to	5.23-04	17	9	9	5	10	6	14	4	3	3	4	1	9	4	5	9	112
			6.8	4.7	3.1	4.2	6.7	4.6	10.8	8.5	5.3	4.2	4.0	3.9	6.2	5.6	5.6	5.9	5.7
4.23-04	to	4.70-04	56	36	34	22	34	23	54	25	22	14	4	8	18	19	22	41	432
			8.5	5.9	4.5	5.2	8.2	5.6	13.0	9.8	6.7	5.3	4.3	4.5	7.5	7.2	7.1	7.9	7.1
3.81-04	to	4.23-04	38	26	7	5	11	11	14	7	8	3	2	3	6	10	21	19	191
			9.7	6.7	4.8	5.4	8.7	6.1	13.6	10.2	7.2	5.5	4.4	4.7	7.9	8.0	8.6	8.8	7.7
3.43-04	to	3.81-04	19	18	7	7	10	8	13	3	3	4	7	3	6	5	13	14	140
			10.3	7.3	5.0	5.7	9.1	6.4	14.2	10.3	7.3	5.9	5.0	4.9	8.4	8.4	9.5	9.5	8.2
3.09-04	to	3.43-04	35	22	6	2	5	7	10	11	8	6	4	7	19	12	7	28	189
			11.4	8.1	5.3	5.8	9.3	6.8	14.6	10.9	7.9	6.3	5.3	5.4	9.7	9.3	10.0	10.9	8.8
2.78-04	to	3.09-04	74	62	56	30	42	51	51	46	38	30	18	28	24	32	20	51	653
			13.7	10.2	7.5	7.2	11.1	9.1	16.7	13.4	10.2	8.6	6.6	7.3	11.4	12.0	11.4	13.4	10.8
2.50-04	to	2.78-04	30	17	4	7	10	9	5	16	11	2	5	5	14	7	10	32	184
			14.6	10.7	7.6	7.5	11.5	9.5	16.9	14.2	10.9	8.8	7.0	7.6	12.4	12.5	12.1	15.0	11.4
2.25-04	to	2.50-04	108	91	55	55	54	69	75	69	41	32	40	23	46	42	46	65	911
			18.0	13.8	9.8	10.0	13.8	12.6	20.1	17.9	13.5	11.2	10.0	9.2	15.7	15.9	15.4	18.2	14.3
2.02-04	to	2.25-04	64	17	21	16	16	13	17	24	18	12	9	8	18	16	18	25	312
			20.0	14.3	10.7	10.8	14.5	13.2	20.8	19.2	14.6	12.2	10.7	9.7	16.9	17.2	16.6	19.4	15.3
1.82-04	to	2.02-04	74	63	39	20	25	32	28	40	21	20	18	27	31	34	34	60	566
			22.3	16.4	12.2	11.7	15.6	14.6	21.9	21.4	16.0	13.7	12.1	11.6	19.1	20.8	19.0	22.3	17.1
1.64-04	to	1.82-04	95	62	72	76	79	60	90	56	60	34	41	46	48	41	32	57	949
			25.3	18.5	15.1	15.1	19.0	17.3	25.7	24.4	19.7	16.3	15.1	14.7	22.5	23.3	21.3	25.1	20.1
1.48-04	to	1.64-04	93	55	43	48	25	21	25	22	30	19	27	35	37	43	40	52	615
			28.2	20.3	16.8	17.3	20.0	18.3	26.8	25.5	21.6	17.8	17.2	17.1	25.2	26.8	24.1	27.1	22.1

TABLE 2.3-30 (Cont'd)

CUMULATIVE FREQUENCY DISTRIBUTION OF X/Q* FOR A 2-HOUR TIME PERIOD

AT THE MINIMUM EXCLUSION AREA BOUNDARY DISTANCE (445 M), BYRON SITE

CHI/Q RANGE GT		CHI/Q RANGE LE	DOWNWIND SECTOR N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
1.33-04	to	1.48-04	41	28	25	19	15	21	18	15	21	21	6	19	22	12	16	23	322
			29.5	21.3	17.7	18.2	20.7	19.2	27.5	26.4	22.9	19.4	17.6	18.4	26.7	27.8	25.2	28.8	23.1
1.20-04	to	1.33-04	155	123	131	131	107	83	97	91	92	64	69	73	83	59	92	108	1558
			34.3	25.4	22.9	24.2	25.2	22.9	31.6	31.2	28.7	24.3	22.8	23.3	32.6	32.6	31.7	34.1	28.0
1.08-04	to	1.20-04	100	57	69	56	55	45	55	34	37	35	41	45	40	37	52	68	826
			37.4	27.3	25.7	26.8	27.6	25.0	33.9	33.0	31.0	27.0	25.9	26.4	35.4	35.6	35.4	37.5	30.6
9.68-05	to	1.08-04	159	140	87	113	86	89	94	83	95	60	83	76	73	46	68	112	1454
			42.4	32.0	29.1	31.9	31.3	29.0	37.8	37.5	37.0	31.6	31.4	31.6	40.6	39.3	40.2	43.0	35.3
8.72-05	to	9.68-05	120	101	71	59	59	72	62	67	53	51	39	46	50	49	46	63	1008
			46.1	35.3	31.9	34.6	33.8	32.2	40.4	41.1	40.3	35.6	34.4	34.7	44.2	43.3	43.4	46.1	38.5
7.84-05	to	8.72-05	117	145	135	110	95	79	93	72	96	91	72	77	56	54	45	88	1425
			49.8	40.2	37.3	39.7	37.8	35.8	44.3	44.9	46.4	42.6	39.8	39.9	48.1	47.7	46.6	50.4	43.0
7.06-05	to	7.84-05	84	84	56	44	41	40	32	35	32	19	29	37	12	26	15	39	625
			52.4	43.0	39.5	41.7	39.6	37.6	45.6	46.8	48.4	44.0	42.0	42.4	49.0	49.8	47.6	52.3	45.0
6.35-05	to	7.06-05	201	177	163	152	131	144	131	91	94	96	79	120	80	76	78	104	1917
			58.7	48.9	46.0	48.6	45.2	44.1	51.1	51.7	54.3	51.4	47.9	50.6	54.6	56.0	53.1	57.4	51.0
5.72-05	to	6.35-05	172	167	167	114	136	125	120	113	98	91	88	74	93	65	71	118	1812
			64.0	54.5	52.6	53.8	51.0	49.7	56.1	57.7	60.4	58.4	54.6	55.6	61.2	61.3	58.1	63.2	56.8
5.15-05	to	5.72-05	134	123	114	74	74	101	100	65	74	61	56	78	70	49	44	55	1272
			68.2	58.6	57.1	57.2	54.1	54.3	60.3	61.2	65.1	63.1	58.8	61.0	66.2	65.3	61.2	65.9	60.8
4.63-05	to	5.15-05	170	146	121	100	106	85	110	87	69	86	74	90	60	47	75	95	1521
			73.5	63.4	61.9	61.8	58.7	58.1	64.9	65.9	69.4	69.7	64.4	67.1	70.4	69.1	66.5	70.6	65.7
4.17-05	to	4.63-05	139	152	139	113	94	118	105	81	80	52	80	64	55	49	55	109	1485
			77.9	68.5	67.4	67.0	62.7	63.4	69.3	70.2	74.4	73.7	70.4	71.4	74.3	73.1	70.4	75.9	70.4
3.75-05	to	4.17-05	145	151	142	125	135	130	94	73	66	70	84	86	52	49	43	67	1512
			82.4	73.6	73.0	72.7	68.4	69.3	73.2	74.1	78.6	79.1	76.7	77.3	78.0	77.1	73.4	79.2	75.2
3.38-05	to	3.75-05	70	87	88	79	98	72	81	60	30	36	43	69	57	44	49	51	1014
			84.6	78.5	76.5	76.3	72.6	72.5	76.6	77.3	80.5	81.9	80.0	82.0	82.1	80.7	76.9	81.7	78.4
0.00	to	3.38-05	494	705	593	518	641	610	560	423	311	235	266	265	253	238	328	372	6812
			100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

*X/Q values, expressed in (sec/m^3), are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.
Key: 1.33-04 = 1.33 x 10^{-4}.

TABLE 2.3-31

5% AND 50% PROBABILITY LEVEL χ/Q AT THE MINIMUM EXCLUSION AREA BOUNDARY DISTANCE (445 M)

DOWNWIND SECTOR	χ/Q* 5% 1 HOUR	5% 2 HOURS	50% 1 HOUR	50% 2 HOURS
N	9.1	6.4	1.1	0.78
NNE	7.7	5.0	0.87	0.62
NE	6.1	3.5	0.88	0.60
ENE	6.5	4.7	0.94	0.62
E	15.	9.5	0.80	0.58
ESE	6.5	4.9	0.78	0.57
SE	22.	13.	0.92	0.65
SSE	18.	12.	0.98	0.66
S	9.1	5.5	1.0	0.69
SSW	6.6	4.7	1.0	0.65
SW	5.2	3.4	0.91	0.62
WSW	5.2	3.5	0.87	0.64
W	9.5	5.8	1.2	0.69
WNW	8.7	5.4	1.1	0.70
NW	8.7	5.3	1.1	0.68
NNW	8.9	5.5	1.1	0.79
All Sectors	9.1	5.7	0.96	0.65

**χ/Q values, expressed in (sec/m^3) x 10^{-4}, are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.

TABLE 2.3-32

CUMULATIVE FREQUENCY DISTRIBUTION OF X/Q* FOR A 8-HOUR TIME PERIOD

AT THE OUTER BOUNDARY OF THE LOW POPULATION ZONE (4828 M), BYRON SITE

CHI/Q RANGE		DOWNWIND SECTOR																
GT	LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
3.34-05 to		151	64	53	76	204	106	291	153	75	53	83	76	102	61	45	78	1671
		2.5	1.0	1.0	1.5	4.0	2.2	6.0	3.9	2.3	1.9	3.0	2.6	3.4	2.2	1.4	1.8	2.5
2.84-05 to	3.34-05	29	21	14	1	22	7	28	32	4	0	8	1	3	5	1	21	197
		2.9	1.4	1.2	1.5	4.4	2.4	6.6	4.7	2.4	1.9	3.3	2.6	3.6	2.4	1.5	2.3	2.8
2.41-05 to	2.84-05	71	45	13	12	12	18	29	16	19	0	7	20	7	8	21	16	314
		4.1	2.1	1.5	1.7	4.7	2.7	7.2	5.1	2.9	1.9	3.6	3.3	3.8	2.6	2.1	2.6	3.3
2.05-05 to	2.41-05	55	31	18	17	26	21	52	25	9	4	3	19	11	7	23	19	340
		5.0	2.6	1.8	2.1	5.2	3.2	8.2	5.8	3.2	2.1	3.7	3.9	4.2	2.9	2.9	3.1	3.8
1.74-05 to	2.05-05	88	64	27	54	74	74	90	91	45	44	11	28	27	38	36	36	827
		6.4	3.6	2.3	3.1	6.6	4.7	10.1	8.1	4.6	3.7	4.1	4.9	5.1	4.3	4.0	3.9	5.0
1.48-05 to	1.74-05	89	51	26	37	41	58	28	39	34	35	3	6	35	17	53	75	627
		7.9	4.5	2.7	3.9	7.4	5.9	10.7	9.1	5.6	5.0	4.2	5.1	6.3	4.9	5.7	5.6	6.0
1.26-05 to	1.48-05	117	73	33	16	18	15	38	34	18	13	2	21	33	20	29	77	557
		9.8	5.6	3.3	4.2	7.8	6.2	11.4	10.0	6.1	5.4	4.3	5.8	7.4	5.6	6.6	7.4	6.8
1.07-05 to	1.26-05	145	94	38	48	44	24	67	64	40	18	8	20	22	31	36	90	789
		12.2	7.2	4.0	5.1	8.6	6.7	12.8	11.6	7.3	6.1	4.5	6.4	8.1	6.7	7.8	9.4	8.0
9.10-06 to	1.07-05	132	84	50	40	56	56	93	76	41	19	32	21	45	43	37	101	926
		14.3	8.5	4.9	5.9	9.7	7.9	14.7	13.5	8.6	6.8	5.7	7.2	9.6	8.2	9.0	11.7	9.4
7.74-06 to	9.10-06	209	119	123	96	96	112	174	86	68	57	48	38	86	68	85	124	1589
		17.7	10.4	7.2	7.8	11.6	10.3	18.3	15.7	10.6	8.9	7.5	8.4	12.5	10.7	11.7	14.6	11.8
6.57-06 to	7.74-06	175	108	90	39	53	60	40	70	37	62	23	33	95	76	63	112	1136
		20.6	12.2	8.8	8.5	12.6	11.5	19.1	17.5	11.7	11.1	8.3	9.6	15.8	13.4	13.7	17.2	13.5
5.59-06 to	6.57-06	172	138	96	68	55	71	112	63	98	55	40	74	65	73	81	151	1412
		23.4	14.4	10.5	9.9	13.7	13.0	21.4	19.1	14.6	13.1	9.7	12.0	18.0	16.0	16.3	20.6	15.6
4.75-06 to	5.59-06	222	177	154	127	133	120	130	145	129	84	90	95	75	95	89	126	1991
		27.0	17.3	13.3	12.3	16.3	15.5	24.1	22.8	18.5	16.2	13.0	15.3	20.5	19.4	19.1	23.5	18.6
4.04-06 to	4.75-06	163	165	95	102	86	56	93	91	92	75	60	77	93	64	77	130	1519
		29.7	19.9	15.1	14.3	18.0	16.7	26.0	25.1	21.3	18.9	15.2	17.9	23.6	21.7	21.6	26.5	20.8

TABLE 2.3-32 (Cont'd)

CUMULATIVE FREQUENCY DISTRIBUTION OF X/Q* FOR A 8-HOUR TIME PERIOD

AT THE OUTER BOUNDARY OF THE LOW POPULATION ZONE (4828 M), BYRON SITE

CHI/Q RANGE			DOWNWIND SECTOR																	
GT		LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
3.43-06	to	4.04-06	253	299	209	151	103	125	140	116	119	98	143	115	80	114	101	171	2337
			33.8	24.8	18.8	17.3	20.0	19.3	28.9	28.1	24.8	22.5	20.4	21.7	26.3	25.8	24.8	30.4	24.4
2.92-06	to	3.43-06	279	289	203	218	206	204	216	191	192	127	122	147	147	145	139	232	3057
			38.3	29.5	22.5	21.5	24.1	23.5	33.3	33.0	30.6	27.1	24.8	26.7	31.3	30.9	29.3	35.7	28.9
2.48-06	to	7.92-06	318	263	254	241	194	152	154	128	122	110	137	143	104	134	124	156	2734
			43.5	33.7	27.1	26.3	27.8	26.7	36.5	36.3	34.3	31.1	29.8	31.5	34.8	35.7	33.2	39.3	33.0
2.11-06	to	2.48-06	348	407	277	243	310	227	238	193	153	150	184	226	190	172	145	199	3662
			49.2	40.3	32.2	31.0	33.9	31.4	41.4	41.2	38.9	36.6	36.5	39.1	41.3	41.9	37.9	43.8	38.5
1.79-06	to	2.11-06	326	218	187	232	201	156	143	145	89	96	147	158	110	77	140	189	2614
			54.5	43.8	35.6	35.5	37.8	34.7	44.3	44.9	41.5	40.1	41.9	44.5	45.0	44.6	42.3	48.2	42.5
1.52-06	to	1.79-06	327	423	404	374	293	289	356	203	231	162	189	186	225	170	222	288	4342
			59.9	50.6	42.9	42.9	43.6	40.7	51.7	50.1	48.5	46.0	48.7	50.8	52.6	50.7	49.4	54.8	49.0
1.29-06	to	1.52-06	242	203	250	165	197	175	164	134	125	142	112	119	121	82	111	198	2540
			63.8	53.9	47.4	46.1	47.4	44.4	55.0	53.5	52.2	51.1	52.8	54.8	56.7	53.6	53.0	59.3	52.8
1.10-06	to	1.29-06	311	327	303	221	282	268	227	199	214	159	184	197	175	115	164	234	3580
			68.9	59.2	52.9	50.4	53.0	50.0	59.7	58.6	58.6	56.9	59.5	61.4	62.6	57.8	58.2	64.7	58.1
9.35-07	to	1.10-06	251	321	354	251	244	257	268	138	179	197	158	199	145	214	170	190	3536
			73.0	64.4	59.3	55.3	57.7	55.3	65.2	62.1	64.0	64.1	65.3	68.1	67.5	65.4	63.7	69.0	63.5
7.95-07	to	9.35-07	300	327	367	368	318	248	217	217	161	110	194	168	141	128	170	246	3680
			77.9	69.7	66.0	62.5	64.0	60.5	69.7	67.6	68.8	68.1	72.3	73.8	72.3	70.0	69.1	74.6	69.0
6.76-07	to	7.95-07	190	201	196	242	201	254	197	151	116	99	90	148	131	117	132	135	2600
			81.0	72.9	69.6	67.2	67.9	65.8	73.7	71.5	72.3	71.7	75.6	78.8	76.7	74.2	73.3	77.7	72.9
5.74-07	to	6.76-07	247	293	298	261	243	212	174	170	179	148	108	129	126	144	96	135	2963
			85.0	77.7	75.0	72.4	72.6	70.3	77.3	75.8	77.7	77.1	79.5	83.2	81.0	79.3	76.4	80.8	77.3
4.88-07	to	5.74-07	173	203	175	246	212	267	103	145	138	110	126	69	85	87	93	119	2351
			87.8	81.0	78.1	77.2	76.8	75.8	79.4	79.5	81.8	81.1	84.1	85.5	83.8	82.4	79.4	83.6	80.8
4.15-07	to	4.88-07	120	173	177	188	138	90	117	116	80	110	71	98	60	49	91	103	1781
			89.8	83.8	81.3	80.8	79.5	77.7	81.8	82.5	84.2	85.1	86.7	88.8	85.9	84.2	82.3	85.9	83.5
0.00	to	4.15-07	626	1005	1028	980	1048	1068	884	686	525	408	366	332	418	442	555	615	10986
			100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

*X/Q values, expressed in (sec/m^3), are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.
Key: 3.43-06 = 3.43 x 10^{-6}.

TABLE 2.3-33

CUMULATIVE FREQUENCY DISTRIBUTION OF X/Q* FOR A 16-HOUR TIME PERIOD

AT THE OUTER BOUNDARY OF THE LOW POPULATION ZONE (4828 M), BYRON SITE

CHI/Q RANGE			DOWNWIND SECTOR																
GT		LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
7.34-06	to		127	24	39	90	198	50	226	144	80	57	82	72	105	35	31	28	1388
			1.5	.3	.5	1.2	2.6	.7	3.2	2.5	1.6	1.4	2.0	1.7	2.3	.8	.6	.4	1.4
6.24-06	to	7.34-06	1	8	38	8	51	2	73	21	7	1	14	23	14	1	1	31	294
			1.5	.4	.9	1.3	3.3	.7	4.2	2.8	1.7	1.4	2.4	2.2	2.7	.8	.7	.9	1.7
5.30-06	to	6.24-06	44	10	20	3	37	18	32	6	10	0	14	15	31	13	10	3	266
			2.0	.5	1.2	1.3	3.8	1.0	4.7	2.9	1.9	1.4	2.7	2.5	3.4	1.1	.9	.9	2.0
4.51-06	to	5.30-06	59	14	4	14	20	9	72	13	22	2	11	16	13	47	21	16	353
			2.7	.6	1.2	1.5	4.0	1.1	5.7	3.1	2.4	1.4	3.0	2.9	3.6	2.2	1.3	1.2	2.3
3.83-06	to	4.51-06	34	15	14	4	23	29	78	14	19	33	16	19	5	5	36	32	376
			3.1	.8	1.4	1.5	4.3	1.5	6.8	3.4	2.8	2.2	3.4	3.4	3.8	2.4	2.1	1.7	2.7
3.26-06	to	3.83-06	35	40	8	53	12	5	85	47	36	30	7	17	19	7	3	22	426
			3.5	1.2	1.5	2.2	4.5	1.6	8.0	4.2	3.5	3.0	3.5	3.7	4.2	2.5	2.1	2.0	3.1
2.77-06	to	3.26-06	114	55	23	42	72	91	95	91	28	0	21	33	19	31	1	37	753
			4.8	1.9	1.8	2.8	5.4	2.9	9.4	5.8	4.1	3.0	4.1	4.5	4.6	3.2	2.1	2.6	3.9
2.35-06	to	2.77-06	156	113	62	34	69	109	93	123	49	32	31	18	45	18	40	60	1052
			6.6	3.1	2.5	3.2	6.3	4.4	10.7	7.9	5.0	3.7	4.8	4.9	5.6	3.7	3.0	3.5	5.0
2.00-06	to	2.35-06	209	107	49	68	85	36	97	38	50	2	16	29	34	5	50	90	1005
			9.0	4.3	3.1	4.1	7.4	4.9	12.0	8.5	6.0	4.7	5.2	5.6	6.4	3.8	4.0	4.9	6.0
1.70-06	to	2.00-06	143	129	76	35	104	39	122	100	39	36	25	47	66	53	115	133	1262
			10.6	5.8	4.0	4.5	8.8	5.5	13.8	10.2	6.8	5.6	5.8	6.7	7.8	5.0	6.4	6.9	7.3
1.44-06	to	1.70-06	254	215	121	81	120	151	119	173	101	49	13	42	103	65	104	123	1834
			13.6	8.2	5.5	5.6	10.4	7.6	15.4	13.2	8.9	6.8	6.2	7.7	10.1	6.5	8.6	8.8	9.1
1.23-06	to	1.44-06	297	236	97	120	102	169	197	169	86	81	36	75	79	86	95	193	2118
			17.0	10.9	6.7	7.1	11.7	10.0	18.2	16.1	10.6	8.7	7.0	9.4	11.9	8.5	10.6	11.7	11.3
1.04-06	to	1.23-06	310	268	175	131	133	116	137	201	188	124	102	98	84	114	81	255	2517
			20.6	13.9	8.8	8.8	13.5	11.6	20.2	19.5	14.4	11.7	9.6	11.7	13.8	11.2	12.3	15.6	13.8
8.87-07	to	1.04-06	397	303	209	246	192	166	291	228	193	210	138	85	126	184	147	221	3336
			25.1	17.3	11.3	12.0	16.0	14.0	24.3	23.4	18.2	16.8	12.9	13.6	16.6	15.5	15.3	19.0	17.2

TABLE 2.3-33 (Cont'd)

CUMULATIVE FREQUENCY DISTRIBUTION OF X/Q* FOR A 16-HOUR TIME PERIOD

AT THE OUTER BOUNDARY OF THE LOW POPULATION ZONE (4828 M), BYRON SITE

CHI/Q RANGE			DOWNWIND SECTOR																
GT		LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
7.54-07	to	8.87-07	466	369	283	304	244	289	339	209	188	215	164	152	197	125	139	272	3955
			30.5	21.4	14.8	15.9	19.2	18.0	29.1	27.0	22.0	21.9	17.0	17.1	21.0	18.4	18.2	23.1	21.2
6.41-07	to	7.54-07	479	376	347	228	230	266	282	194	216	137	189	247	216	182	206	383	4178
			36.0	25.6	19.0	18.9	22.2	21.8	33.1	30.3	26.3	25.2	21.6	22.9	25.8	22.6	22.5	29.0	25.4
5.45-07	to	6.41-07	490	564	489	392	363	371	314	273	207	249	271	319	302	248	233	386	5471
			41.7	32.0	24.9	23.9	27.0	27.0	37.5	35.0	30.5	31.2	28.3	30.2	32.6	28.4	27.4	34.8	30.9
4.63-07	to	5.45-07	512	579	376	320	392	394	387	302	311	198	215	248	300	188	201	338	5261
			47.6	38.5	29.5	28.1	32.1	32.6	43.0	40.2	36.7	36.0	33.6	36.0	39.3	32.8	31.6	40.0	36.3
3.94-07	to	4.63-07	546	545	502	403	390	377	305	344	276	216	273	275	214	255	416	368	5705
			53.9	44.6	35.6	33.3	37.2	37.9	47.3	46.0	42.3	41.2	40.3	42.4	44.1	38.7	40.2	45.6	42.0
3.35-07	to	3.94-07	429	440	423	385	347	293	264	222	159	161	288	210	228	217	173	351	4590
			58.8	49.6	40.7	38.2	41.8	42.0	51.0	49.8	45.5	45.1	47.4	47.2	49.2	43.7	43.8	51.0	46.7
2.84-07	to	3.35-07	570	511	534	484	548	373	494	391	294	218	312	379	371	362	306	463	6610
			65.4	55.3	47.2	44.5	48.9	47.3	58.0	56.5	51.4	50.3	55.1	56.0	57.4	52.2	50.2	58.0	53.4
2.42-07	to	2.84.07	406	421	375	401	264	368	302	163	199	195	182	190	189	239	250	292	4436
			70.1	60.0	51.7	49.7	52.4	52.5	62.3	59.3	55.4	55.0	59.6	60.4	61.7	57.7	55.4	62.5	57.9
2.06-07	to	2.42-07	389	544	679	557	458	401	462	256	267	289	282	314	293	324	431	466	6412
			74.5	66.1	60.0	56.9	58.4	58.1	68.8	63.7	60.7	62.0	66.5	67.6	68.2	65.3	64.4	69.6	64.3
1.75-07	to	2.06-07	364	436	364	349	419	350	303	245	330	179	251	254	233	202	322	357	4958
			78.7	71.0	64.4	61.4	63.9	63.0	73.1	67.9	67.4	66.3	72.7	73.5	73.4	70.0	71.1	75.0	69.4
1.48-07	to	1.75-07	291	383	491	378	434	368	280	311	349	211	172	231	245	292	277	217	4930
			82.1	75.4	70.3	66.3	69.6	68.2	77.1	73.2	74.4	71.3	76.9	78.9	78.9	76.8	76.9	78.3	74.3
1.26-07	to	1.48-07	397	292	471	455	396	376	210	173	213	195	160	219	140	174	155	246	4272
			86.7	78.6	76.0	72.1	74.8	73.5	80.1	76.2	78.6	76.0	80.9	83.9	82.0	80.8	80.1	82.1	78.7
1.07-07	to	1.26-07	174	225	241	308	227	135	109	170	125	112	181	89	117	96	53	168	2530
			88.7	81.2	79.0	76.1	77.8	75.4	81.6	79.1	81.1	78.7	85.3	86.0	84.6	83.1	81.2	84.6	81.2
9.12-08	to	1.07-07	209	313	270	248	279	411	237	269	190	256	122	132	123	172	165	213	3609
			91.1	84.7	82.2	79.3	81.4	81.2	84.9	83.7	85.0	84.9	88.3	89.1	87.4	87.1	84.7	87.9	84.9
0.00	to	9.12-08	774	1363	1465	1601	1416	1331	1064	953	749	628	474	473	565	556	735	794	14941
			100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

*X/Q values, expressed in (sec/m^3), are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.
Key: 7.54-07 = 7.54 x 10^{-7}.

TABLE 2.3-34

CUMULATIVE FREQUENCY DISTRIBUTION OF X/Q* FOR A 72-HOUR TIME PERIOD

AT THE OUTER BOUNDARY OF THE LOW POPULATION ZONE (4828 M), BYRON SITE

CHI/Q RANGE			DOWNWIND SECTOR																
GT		LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
3.67-06	to		208	0	0	73	323	70	318	355	140	141	157	0	236	74	63	0	2158
			1.3	.0	.0	.5	2.1	.5	2.1	2.7	1.2	1.4	1.6	.0	2.3	.7	.5	.0	1.0
3.12-06	to	3.67-06	22	40	0	72	37	67	145	6	129	9	5	70	5	1	6	0	609
			1.4	.2	.0	.9	2.3	.9	3.1	2.7	2.3	1.5	1.7	.7	2.3	.7	.6	.0	1.3
2.65-06	to	3.12-06	80	30	0	0	89	2	197	11	70	2	1	14	1	1	3	4	505
			1.9	.4	.0	.9	2.9	.9	4.5	2.8	2.9	1.5	1.7	.8	2.3	.7	.6	.0	1.5
2.25-06	to	2.65-06	75	1	104	26	73	31	91	126	2	1	2	151	58	70	0	66	877
			2.4	.4	.6	1.1	3.3	1.2	5.1	3.7	2.9	1.5	1.7	2.4	2.9	1.4	.6	.5	2.0
1.92-06	to	2.25-06	128	69	44	151	204	48	128	32	2	6	42	122	35	2	71	26	1110
			3.2	.9	.9	2.0	4.7	1.5	5.9	4.0	2.9	1.6	2.1	3.6	3.2	1.4	1.2	.7	2.5
1.63-06	to	1.92-06	199	80	156	108	148	12	329	169	5	58	157	61	121	0	2	132	1737
			4.4	1.3	1.9	2.7	5.6	1.6	8.2	5.2	3.0	2.1	3.7	4.2	4.4	1.4	1.2	1.6	3.3
1.38-06	to	1.63-06	182	94	80	16	185	8	194	124	3	1	3	14	55	26	68	163	1216
			5.5	1.9	2.4	2.8	6.8	1.6	9.5	6.2	3.0	2.1	3.8	4.3	4.9	1.6	1.8	2.8	3.9
1.18-06	to	1.38-06	388	45	100	52	144	177	83	64	118	17	29	72	94	137	7	162	1689
			7.9	2.2	3.0	3.2	7.7	2.8	10.0	6.6	4.0	2.3	4.1	5.1	5.8	2.9	1.9	3.9	4.7
1.00-06	to	1.18-06	333	148	60	67	115	138	380	144	87	116	65	71	129	168	135	179	2335
			10.0	3.1	3.4	3.6	8.5	3.8	12.6	7.7	4.7	3.4	4.7	5.8	7.1	4.5	3.0	5.2	5.8
8.50-07	to	1.00-06	449	243	136	117	208	156	300	163	268	155	73	134	141	65	150	233	2991
			12.7	4.6	4.2	4.3	9.8	4.8	14.6	8.9	7.0	5.0	5.5	7.1	8.5	5.1	4.3	6.9	7.2
7.22-07	to	8.50-07	592	582	148	448	342	337	439	252	128	145	46	230	222	77	70	212	4270
			16.4	8.1	5.1	7.2	12.0	7.1	17.6	10.8	8.1	6.4	5.9	9.4	10.6	5.8	4.9	8.4	9.2
6.14-07	to	7.22-07	772	538	466	301	272	385	268	516	301	65	255	230	167	195	135	299	5135
			21.1	11.4	8.0	9.1	13.8	9.8	19.4	14.7	10.6	7.0	8.3	11.8	12.2	7.7	6.0	10.5	11.7
5.22-07	to	6.14-07	678	595	501	327	446	280	527	423	280	214	144	142	70	95	333	405	5460
			25.3	15.0	11.1	11.1	16.7	11.7	22.9	17.9	13.0	9.1	9.7	13.2	12.9	8.6	8.8	13.4	14.2
4.44-07	to	5.22-07	705	824	240	537	501	410	576	398	278	331	148	240	113	204	457	552	6514
			29.7	20.0	12.6	14.5	19.9	14.5	26.8	20.8	15.4	12.4	11.3	15.6	14.0	10.5	12.7	17.3	17.3

TABLE 2.3-34 (Cont'd)

CUMULATIVE FREQUENCY DISTRIBUTION OF X/Q* FOR A 72-HOUR TIME PERIOD

AT THE OUTER BOUNDARY OF THE LOW POPULATION ZONE (4828 M), BYRON SITE

CHI/Q RANGE			DOWNWIND SECTOR																
GT		LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
3.77-07	to	4.44-07	805	1107	577	397	293	547	579	709	257	305	259	328	270	142	426	603	7604
			34.6	26.8	16.2	17.0	21.8	18.2	30.7	26.1	17.5	15.4	13.9	18.9	16.6	11.8	16.3	21.6	20.9
3.21-07	to	3.77-07	1004	1030	640	601	493	824	361	493	473	305	212	381	299	535	304	644	8599
			40.8	33.1	20.2	20.9	25.0	23.9	33.2	29.8	21.6	18.4	16.1	22.7	19.5	16.9	18.8	26.2	25.0
2.72-07	to	3.21-07	926	857	693	603	615	735	622	492	489	707	378	351	490	379	402	692	9431
			46.5	38.3	24.5	24.7	28.9	28.9	37.4	33.5	25.7	25.3	20.0	26.3	24.2	20.4	22.2	31.1	29.4
2.32-07	to	2.72-07	806	998	941	845	631	765	727	613	861	425	400	318	545	391	378	926	10570
			51.5	44.4	30.3	30.0	33.0	34.1	42.3	38.1	33.0	29.5	24.1	29.5	29.5	24.1	25.4	37.7	34.4
1.97-07	to	2.32-07	1099	1142	1051	933	871	872	935	605	529	493	699	448	533	491	771	880	12352
			58.3	51.3	36.9	35.9	38.6	40.1	48.6	42.6	37.5	34.3	31.2	34.0	34.6	28.8	31.9	43.9	40.3
1.67-07	to	1.97-07	896	921	975	1037	901	1025	829	647	334	713	722	476	593	482	625	748	11924
			63.8	56.9	42.9	42.5	44.5	47.1	54.2	47.5	40.3	41.3	38.6	38.8	40.4	33.3	37.2	49.2	45.9
1.42-07	to	1.67-07	731	651	885	724	709	687	493	381	474	521	661	468	626	759	697	941	10408
			68.3	60.9	48.4	47.1	49.0	51.8	57.5	50.3	44.4	46.5	45.4	43.5	46.4	40.4	43.1	55.9	50.9
1.21-07	to	1.42-07	706	975	1066	661	664	604	512	660	948	464	634	635	644	696	769	989	11627
			72.7	66.8	55.0	51.3	52.3	55.9	61.0	55.3	52.4	51.0	51.9	49.8	52.6	47.0	49.6	63.0	46.4
1.03-07	to	1.21-07	655	731	831	789	912	554	610	654	804	579	564	370	805	862	511	755	10986
			76.7	71.3	60.2	56.3	59.2	59.7	65-1	60.2	59.2	56.7	57.7	53.6	60.4	55.1	53.9	68.3	61.6
8.74-08	to	1.03-07	694	419	774	905	732	523	342	582	536	426	466	533	445	555	756	662	9350
			81.0	73.8	65.0	62.0	63.9	63.2	67.4	64.5	63.8	60.9	62.5	58.9	64.7	60.3	60.3	73.0	66.0
7.42-08	to	8.74-08	353	550	754	580	835	543	615	377	284	376	615	724	584	633	451	381	8655
			83.2	77.2	69.7	65.7	69.3	67.0	71.5	67.3	66.2	64.6	68.8	66.2	70.3	66.3	64.1	75.7	70.1
6.31-08	to	7.42-08	387	498	776	648	768	375	827	538	652	272	540	640	611	618	416	756	9322
			85.6	80.2	74.5	69.8	74.3	69.5	77.1	71.4	71.7	67.3	74.3	72.6	76.2	72.1	67.6	81.1	74.5
5.36-08	to	6.31-08	359	349	541	814	405	545	494	395	348	289	329	388	136	481	451	285	6609
			87.8	82.4	77.8	74.9	76.9	73.2	80.4	74.3	74.7	70.1	77.7	76.5	77.6	76.6	71.4	83.1	77.6
4.56-08	to	5.36-08	395	364	646	685	449	539	521	479	399	488	507	307	319	427	753	350	7628
			90.2	84.6	81.8	79.3	79.8	76.9	84.0	77.9	78.1	74.9	82.9	79.6	80.6	80.7	77.8	85.6	81.2
0.00	to	4.56-08	1585	2533	2924	3276	3130	3377	2378	2951	2586	2557	1668	2025	2004	2053	2634	2020	39702
			100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

*X/Q values, expressed in (sec/m^3), are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.
Key: 3.77-07 = 3.77 x 10^{-7}.

TABLE 2.3-35

CUMULATIVE FREQUENCY DISTRIBUTION OF X/Q* FOR A 624-HOUR TIME PERIOD

AT THE OUTER BOUNDARY OF THE LOW POPULATION ZONE (4828 M), BYRON SITE

CHI/Q RANGE			DOWNWIND SECTOR																
GT		LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
1.10-06	to		560	0	0	575	744	0	840	1126	0	627	0	0	693	620	0	0	5785
			3.9	.0	.0	4.0	5.2	.0	5.9	7.9	.0	4.4	.0	.0	4.9	4.4	.0	.0	2.5
9.36-07	to	1.10-06	753	0	0	45	893	0	563	145	0	1	0	0	486	2	0	0	2888
			9.2	.0	.0	4.4	11.5	.0	9.8	8.9	.0	4.4	.0	.0	8.3	4.4	.0	.0	3.8
7.95-07	to	9.36-07	804	0	0	3	509	0	264	101	3	1	0	0	95	1	0	0	1781
			14.9	.0	.0	4.4	15.1	.0	11.7	9.6	.0	4.5	.0	.0	8.9	4.4	.0	.0	4.6
6.76-07	to	7.95-07	440	0	0	1	337	619	160	563	252	2	47	0	126	1	48	0	2596
			17.9	.0	.0	4.4	17.4	4.3	12.8	13.6	1.8	4.5	.3	.0	9.8	4.4	.3	.0	5.7
5.75-07	to	6.76-07	319	102	0	111	331	3	944	430	655	1	547	90	223	272	498	73	4599
			20.2	.7	.0	5.2	19.7	4.4	19.4	16.6	6.4	4.5	4.2	.7	11.4	6.3	3.8	.5	7.8
4.88-07	to	5.75-07	1015	479	466	815	701	150	658	597	551	1	394	315	1	119	74	468	6804
			27.3	4.1	3.3	10.9	24.7	5.4	24.1	20.8	10.3	4.5	6.9	3.0	11.4	7.1	4.4	3.8	10.8
4.15-07	to	4.88-07	1661	994	476	529	762	470	1061	620	1106	215	170	398	437	234	4	677	9814
			39.0	11.1	6.6	14.6	30.0	8.7	31.5	25.1	18.0	6.0	8.1	5.9	14.5	8.8	4.4	8.6	15.1
3.53-07	to	4.15-07	1016	1148	1214	691	749	1058	1189	534	530	61	187	1292	603	3	16	764	11055
			46.1	19.1	15.1	19.4	35.3	16.1	39.8	28.9	21.7	6.4	9.4	15.4	18.7	8.8	4.5	13.9	19.9
3.00-07	to	3.53-07	924	1509	891	669	1817	623	1721	728	48	404	153	303	247	1	703	1293	12034
			52.6	29.7	21.4	24.1	48.0	20.5	51.9	34.0	22.1	9.3	10.5	17.7	20.4	8.8	9.4	23.0	25.2
2.55-07	to	3.00-07	1328	2246	738	857	875	860	1009	881	269	258	752	366	283	241	787	1282	13032
			61.9	45.5	26.6	30.1	54.2	26.5	59.0	40.2	24.0	11.1	15.8	20.4	22.4	10.5	14.9	32.0	31.0
2.17-07	to	2.55-07	701	2631	1235	681	632	1161	1252	420	539	798	757	1032	679	615	818	1570	15521
			66.8	63.9	35.2	34.9	58.6	34.7	67.8	43.1	27.7	16.8	21.1	28.0	27.2	14.8	20.7	43.1	37.8
1.84-07	to	2.17-07	1435	1152	1890	902	870	1085	974	830	810	311	712	343	297	574	1036	973	14194
			76.9	72.0	48.5	41.3	64.7	42.3	74.6	48.9	33.4	19.0	26.1	30.5	29.3	18.8	28.0	49.9	44.1
1.57-07	to	1.84-07	1354	1343	2242	1579	414	2219	827	734	1496	467	283	564	657	523	888	947	16537
			86.4	81.4	64.2	52.3	67.6	57.9	80.4	54.1	43.9	22.3	28.1	34.6	33.9	22.5	34.2	56.6	51.3
1.33-07	to	1.57-07	802	1725	1492	942	537	1517	849	1803	1152	295	489	856	1004	364	1197	1638	16662
			92.0	93.5	74.7	58.9	71.4	68.5	86.4	66.7	52.0	24.4	31.5	40.9	40.9	25.0	42.6	68.1	68.7

TABLE 2.3-35 (Cont'd)

CUMULATIVE FREQUENCY DISTRIBUTION OF X/Q* FOR A 624-HOUR TIME PERIOD

AT THE OUTER BOUNDARY OF THE LOW POPULATION ZONE (4828 M), BYRON SITE

CHI/Q RANGE			DOWNWIND SECTOR																	
GT		LE	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	ALL
1.13-07	to	1.33-07	479	678	1091	1372	990	1338	656	1816	1789	1164	818	1791	838	1450	978	915	18163
			95.4	98.3	82.3	68.6	78.3	77.9	91.0	79.5	64.6	32.6	37.3	54.1	46.8	35.2	49.4	74.5	66.7
9.62-08	to	1.13-07	427	165	997	1236	861	1056	518	964	1205	1298	1436	907	529	1170	1566	730	15065
			98.4	99.4	89.3	77.2	84.4	85.3	94.6	86.2	73.0	41.8	47.3	60.8	50.5	43.4	60.4	79.7	73.3
8.17-08	to	9.62-08	55	71	527	1156	1111	911	263	503	1060	1640	1266	967	720	1247	1296	747	13540
			98.7	99.9	93.0	85.3	92.1	91.7	96.5	89.8	80.4	53.4	56.2	67.9	55.6	52.2	69.5	84.9	79.3
6.95-08	to	8.17-08	8	9	97	1280	222	647	160	416	242	951	553	797	1105	1179	753	822	9241
			98.8	100.0	93.7	94.3	93.7	96.2	97.6	92.7	82.1	60.2	60.1	73.8	63.3	60.5	74.8	90.7	83.3
5.91-08	to	6.95-08	89	0	276	286	179	259	37	439	656	546	1037	642	794	824	1088	434	7586
			99.4	100.0	95.6	96.3	95.0	98.1	97.8	95.8	86.7	64.0	67.4	78.5	68.9	66.2	82.4	93.7	86.7
5.02-08	to	5.91-08	35	0	307	258	365	84	128	78	686	718	1195	195	729	967	280	280	6305
			99.7	100.0	97.8	98.1	97.5	98.7	98.7	96.3	91.6	69.1	75.8	80.0	74.0	73.0	84.4	95.7	89.4
4.27-08	to	5.02-08	47	0	299	128	284	115	2	253	320	918	578	505	1246	1185	658	310	6848
			100.0	100.0	99.9	99.0	99.5	99.5	98.8	98.1	93.8	75.6	79.8	83.7	82.7	81.3	89.0	97.9	92.5
3.63-08	to	4.27-08	0	0	14	12	69	8	64	2	225	1444	295	528	559	783	807	170	4980
			100.0	100.0	100.0	99.1	100.0	99.5	99.2	98.1	95.4	85.8	81.9	87.6	86.7	86.8	94.7	99.1	94.6
3.08-08	to	3.63-08	0	0	0	95	0	14	113	32	126	512	265	142	415	383	240	8	2345
			100.0	100.0	100.0	99.8	100.0	99.6	100.0	98.3	96.3	89.5	83.7	88.6	89.6	89.5	96.4	99.1	95.7
2.62-08	to	3.08-08	0	0	0	29	0	10	0	144	296	624	68	129	13	415	123	67	1918
			100.0	100.0	100.0	100.0	100.0	99.7	100.0	99.3	98.3	93.9	84.2	89.6	89.7	92.4	97.2	99.6	96.5
2.23-08	to	2.62-08	0	0	0	0	0	1	0	1	7	101	504	45	390	602	110	0	1761
			100.0	100.0	100.0	100.0	100.0	99.7	100.0	99.4	98.4	94.6	87.7	89.9	92.4	96.7	98.0	99.6	97.3
1.89-08	to	2.23-08	0	0	0	0	0	1	0	5	20	359	401	94	443	231	143	0	1697
			100.0	100.0	100.0	100.0	100.0	99.7	100.0	99.4	98.5	97.1	90.6	90.6	95.5	98.3	99.0	99.6	98.0
1.61-08	to	1.89-08	0	0	0	0	0	0	0	0	32	129	23	49	25	183	42	0	483
			100.0	100.0	100.0	100.0	100.0	99.7	100.0	99.4	98.8	98.0	90.7	91.0	95.7	99.6	99.3	99.6	98.3
1.37-08	to	1.61-08	0	0	0	0	0	15	0	2	9	11	554	2	112	9	45	43	802
			100.0	100.0	100.0	100.0	100.0	99.8	100.0	99.4	98.8	98.1	94.6	91.0	96.5	99.6	99.6	99.9	98.6
0.00	to	1.37-08	0	0	0	0	0	28	0	85	168	265	768	1225	503	54	54	12	3162
			100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

*X/Q values, expressed in (sec/m^3), are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.
Key: 1.13-07 = 1.13 x 10^{-7}.

TABLE 2.3-36

MAXIMUM χ/Q AT THE OUTER BOUNDARY OF

THE LOW POPULATION ZONE (4828 M)

BYRON SITE

DOWNWIND SECTOR	χ/Q*			
	8 HOURS	16 HOURS	72 HOURS	624 HOURS
N	29.	2.2	.66	.16
NNE	13.	1.4	.34	.066
NE	13.	1.1	.25	.052
ENE	45.	4.0	.90	.12
E	57.	4.5	1.2	.17
ESE	26.	2.7	.62	.078
SE	50.	4.7	1.1	.38
SSE	57.	4.5	1.0	.21
S	25.	2.2	.50	.080
SSW	74.	8.5	1.9	.25
SW	28.	2.4	.56	.068
WSW	17.	1.5	.36	.064
W	42.	3.9	1.3	.19
WNW	57.	4.5	1.1	.14
NW	15.	1.5	.44	.069
NNW	12.	.96	.27	.059
All Sectors	74.	8.5	1.9	.38

χ/Q values, expressed in (sec/m^3) x 10^{-5}, are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.

TABLE 2.3-37

5% PROBABILITY LEVEL χ/Q AT THE OUTER BOUNDARY

OF THE LOW POPULATION ZONE (4828 M)

BYRON SITE

DOWNWIND SECTOR	χ/Q* 8 HOURS	16 HOURS	72 HOURS	624 HOURS
N	20.	2.7	1.5	1.0
NNE	14.	1.8	0.82	0.48
NE	9.2	1.5	0.74	0.44
ENE	11.	1.6	0.82	0.60
E	23.	3.1	1.8	1.1
ESE	16.	2.0	0.83	0.54
SE	40.	5.1	2.4	1.6
SSE	28.	3.1	1.7	1.5
S	16.	2.3	0.97	0.61
SSW	15.	1.9	0.84	0.45
SW	10.	2.3	0.98	0.55
WSW	16.	2.4	1.2	0.43
W	17.	2.7	1.4	1.1
WNW	14.	1.7	0.92	0.68
NW	16.	1.9	0.70	0.36
NNW	16.	2.0	1.0	0.48
All Sectors	17.	2.4	1.1	0.76

*χ/Q values, expressed in (sec/m^3) x 10^{-6}, are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.

TABLE 2.3-38

50% PROBABILITY LEVEL χ/Q AT THE OUTER BOUNDARY

OF THE LOW POPULATION ZONE (4828 M)

BYRON SITE

DOWNWIND	χ/Q*			
SECTOR	8 HOURS	16 HOURS	72 HOURS	624 HOURS
N	21	4.4	2.4	3.2
NNE	15	3.3	2.0	2.5
NE	12	2.6	1.4	1.8
ENE	11	2.4	1.3	1.6
E	12	2.7	1.4	2.9
ESE	11	2.6	1.5	1.7
SE	16	3.5	1.9	3.1
SSE	15	3.3	1.5	1.8
S	14	3.0	1.3	1.4
SSW	13	2.9	1.3	0.86
SW	15	3.2	1.3	0.92
WSW	16	3.2	1.2	1.2
W	16	3.3	1.3	0.98
WNW	16	3.0	1.1	0.85
NW	15	2.9	1.2	1.1
NNW	17	3.5	1.6	1.8
All Sectors	15	3.1	1.5	1.6

*χ/Q values, expressed in (sec/m^3) x 10^{-7}, are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.

TABLE 2.3-39

ANNUAL AVERAGE χ/Q AT THE ACTUAL

BYRON SITE BOUNDARY

DOWNWIND SECTOR	ACTUAL SITE BOUNDARY (km)	χ/Q*
N	1.88	2.1
NNE	0.72	5.7
NE	0.50	8.3
ENE	0.43	8.4
E	0.42	11.
ESE	0.43	9.1
SE	0.53	11.
SSE	0.80	3.4
S	0.95	1.8
SSW	0.98	1.3
SW	1.07	1.3
WSW	1.21	1.2
W	1.19	1.3
WNW	1.23	1.2
NW	1.13	1.4
NNW	1.04	2.8

*χ/Q values, expressed in (sec/m^3) x 10^{-7}, are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.

TABLE 2.3-40

ANNUAL AVERAGE χ/Q AT VARIOUS DISTANCES FROM THE BYRON STATION

DOWNWARD	χ/Q*									
SECTOR	0.5 MILES	1.5 MILES	2.5 MILES	3.5 MILES	4.5 MILES	7.5 MILES	15 MILES	25 MILES	35 MILES	45 MILES
N	66.	16.	9.4	6.6	5.0	2.8	1.2	0.67	0.44	0.33
NNE	48.	13.	8.0	5.8	4.4	2.5	1.1	0.61	0.40	0.30
NE	39.	11.	6.8	4.8	3.7	2.1	0.90	0.48	0.32	0.23
ENE	31.	9.0	5.8	4.3	3.3	1.9	0.82	0.44	0.29	0.21
E	40.	11.	6.7	4.8	3.6	2.0	0.86	0.46	0.30	0.22
ESE	35.	10.	6.4	4.6	3.5	2.0	0.87	0.46	0.30	0.22
SE	54.	13.	7.7	5.4	4.1	2.3	0.99	0.53	0.35	0.25
SSE	33.	8.7	5.4	3.9	3.0	1.7	0.73	0.39	0.26	0.19
S	23.	6.5	4.2	3.1	2.4	1.4	0.62	0.34	0.23	0.17
SSW	17.	5.2	3.4	2.5	1.9	1.1	0.49	0.26	0.17	0.13
SW	19.	5.6	3.6	2.7	2.1	1.2	0.52	0.28	0.19	0.14
WSW	22.	6.2	3.9	2.8	2.1	1.2	0.51	0.27	0.18	0.13
W	23.	6.2	3.9	2.8	2.1	1.2	0.53	0.29	0.19	0.14
WNW	21.	5.6	3.4	2.4	1.0	1.0	0.45	0.25	0.17	0.12

TABLE 2.3-40 (Cont'd)

DOWNWARD SECTOR	χ/Q* 0.5 MILES	1.5 MILES	2.5 MILES	3.5 MILES	4.5 MILES	7.5 MILES	15 MILES	25 MILES	35 MILES	45 MILES
NW	22.	6.0	3.7	2.6	2.0	1.1	0.50	0.27	0.18	0.13
NNW	41.	10.	6.2	4.4	3.3	1.8	0.79	0.42	0.28	0.20

*χ/Q values, expressed in (sec/m^3) x 10^{-8}, are based on hourly onsite meteorological data for the period of record January 1974 - December 1976.

Table 2.3-41 has been deleted intentionally

Table 2.3-42 has been deleted intentionally

TABLE 2.3-43

AVERAGE MONTHLY TEMPERATURES

(Values in °F)

MONTH	BYRON*	PEORIA**	SPRINGFIELD**	ROCKFORD**
January	21.3	23.8	26.7	20.2
February	27.2	27.7	30.4	24.0
March	34.5	37.3	39.4	34.1
April	47.6	51.3	53.1	48.2
May	58.5	61.5	63.4	58.8
June	67.9	71.3	72.9	68.8
July	72.8	75.1	76.1	72.8
August	69.1	73.5	74.4	71.5
September	58.9	65.5	67.2	63.3
October	49.8	55.0	56.6	52.7
November	36.8	39.9	41.9	37.6
December	24.9	28.0	30.5	24.9
Year	47.3	50.8	52.7	48.1

* Values from Table 2.3-10.

** Values from Local Climatological Data for period 1941-1970 (30-year normal).

TABLE 2.3-44

AVERAGE MONTHLY RELATIVE HUMIDITY

(Value in %)

MONTH	BYRON*	PEORIA*	SPRINGFIELD**	ROCKFORD***
January	87	74	74	75
February	79	73	74	72
March	79	72	72	71
April	63	65	65	69
May	64	67	65	67
June	72	67	66	68
July	72	71	69	70
August	*	73	74	73
September	74	73	73	73
October	62	70	69	70
November	66	75	74	75
December	71	78	79	79
Year	72	71	71	72

* Values from Table 2.3-12.

** Values from Local Climatological Data, 1977, for period 1960-1977.

*** Values from Local Climatological Data, 1976 for period 1964-1976.

TABLE 2.3-45

LONG-TERM TEMPERATURE DATA FOR ROCKFORD

(Values in °F)

MONTH	AVERAGE (1941-1970)	MAXIMUM (1951-1976)	MINIMUM (1951-1976)
January	20.2	60	-22
February	24.0	68	-22
March	34.1	76	-11
April	48.2	87	9
May	58.8	95	24
June	68.8	99	38
July	72.8	103	43
August	71.5	101	41
September	63.3	102	28
October	52.7	90	15
November	37.6	76	-6
December	24.9	65	-20
YEAR	48.1	103	-22

TABLE 2.3-46

LONG-TERM PRECIPITATION (WATER EQUIVALENT)

AVERAGE AND EXTREMES AT ROCKFORD

(Value in inches)

MONTH	AVERAGE TOTAL (1941-1970)	MONTHLY MAXIMUM (1951-1976)	MONTHLY MINIMUM (1951-1976)
January	1.79	4.66	0.18
February	1.29	2.67	0.04
March	2.65	5.62	0.52
April	3.85	9.92	1.79
May	3.86	6.98	1.44
June	4.42	9.98	1.45
July	4.27	11.81	1.30
August	3.66	9.27	0.67
September	4.00	10.68	0.35
October	2.85	8.32	0.01
November	2.37	4.83	0.38
December	1.71	5.04	0.37
YEAR	36.72	11.81	0.01

TABLE 2.3-47

ICE PELLET AND SNOW PRECIPITATION AT ROCKFORD

(Values in inches of ice and/or snow)

MONTH	AVERAGE (1937-1976)	MONTHLY MAXIMUM (1951-1976)	24-HOUR MAXIMUM (1951-1976)
January	7.8	21.4	9.3
February	6.1	19.5	10.9
March	7.3	22.7	10.4
April	1.3	6.7	6.7
May	T	1.0	T
June	0.0	0.0	0.0
July	0.0	0.0	0.0
August	0.0	0.0	0.0
September	0.0	0.0	0.0
October	0.1	2.2	2.2
November	2.8	14.7	9.5
December	8.0	20.0	8.0
YEAR	33.4	22.7	10.9

Note: T equals trace (<0.01 inch)

TABLE 2.3-48

SHORT-TERM RELATIVE HUMIDITY VALUES AT BYRON STATION

(Values in %)

MONTH	YEAR	AVERAGE	MAXIMUM	MINIMUM
January	1977	73.8	100.0	32.4
February	1977	64.3	100.0	26.7
March	1974	67.0	100.0	23.2
April	1974	59.9	95.8	21.6
May	1974	68.6	99.6	34.9
June	1975	70.4	100.0	30.5
July	1975	70.7	100.0	6.3
August	1975	81.0	100.0	46.5
September	1975	56.8	79.9	24.5
October	1975	48.6	79.9	19.1
November	1976	71.1	100.0	22.7
December	1976	78.3	100.0	37.5
Yearly		68.7	100.0	6.3

TABLE 2.3-49

METEOROLOGICAL CONDITIONS ASSOCIATED WITH PERSISTENCE OF

EXTREMELY STABLE "G" STABILITY FOR GREATER THAN 10 HOURS AT BYRON

STARTING DATE	HOURS OF PERSISTENCE	WIND SPEED (IN MPH)	WIND DIRECTION (IN DEGREES)	CLOUD COVER* (IN TENTHS)	SYNOPTIC FLOW PATTERN
12/12/74	11	2.5	165-350	0	Weak cold front approaching from northwest
9/3/74	11	2	300	6	High pressure center over area
10/25/74	11	1.5	219-010	3	Recent cold frontal passage
10/26/74	13	5	205	2	High pressure system approaching from southwest
11/8/74	12	5	175	0	Cold front approaching from northwest
9/22/74	11	2	Shifting	4	Weak cold front to northwest
10/9/75	11	5	230	0	Frontolysis over area
2/3/76	11	2	Shifting	0	High pressure center over area
10/1/76	11	4	Shifting	0	Cold front over area

*Cloud cover estimated from Chicago Midway data.

TABLE 2.3-50

MAINTENANCE LOG FOR

DATA COLLECTION AND RECORDING SYSTEM

Byron Routine Maintenance Checks - May 1, 1973 through April 30, 1975.

DATE	FINDINGS
1 May 73	1. Equipment startup.
20 January 74	1. Replaced Delta-T circuit board card (later in October 1974 this card was found to have a range of ±3°C rather than ±5°C as was intended). All Delta-T's were appropriately adjusted and the record is now accurate. This finding was reported to the NRC as soon as it was discovered and is now considered closed.
	2. Replaced "side marker" in the wind run recorder.
4 February 74	1. Replaced 540□ azimuth pot #ES12572 on 30' WD.
	2. Changed magnetic gear assembly #13608 on 250' WD.
14 June 74	1. All equipment appeared to be working properly.
16 July 74	1. 250' relative humidity junction box leaked and corrosion had formed. Box was repaired and cleaned.
	2. 250' WD was not switching properly to pot B causing chart to switch erratically. Found field effect transistor 02 to be defective. It was replaced.
7 October 74	1. Found relative humidity sensors at both 30' and 250' levels to need replacement.
	2. 30' RH pulse transmitter was pulled and sent to Westinghouse for repair.

TABLE 2.3-50 (Cont'd)

DATE		FINDINGS
28 October 74	1.	Recalibrated Delta-T sensors and checked W tape recorder.
22 January 75	1.	Changed 30' and 250' relative humidity sensors.
	2.	30' RH pulse transmitter still out for repair.
15 April 75	1.	Readjust magnetic gear #1074-1 of 30' WD sensor.

Byron Bimonthly Routine Maintenance Checks - May 1, 1975 through December 31, 1976.

DATE		FINDINGS
6 June 75	1.	30 ft. relative humidity sensor out of calibration. No calibration data at site. MRI to be contacted for data.
28 August 75	1.	Differential temperature recording 1□F low - corrected.
	2.	Rain gauge side marker off pawl - corrected.
	3.	30' wind run side marker intermittent - to be replaced.
	4.	250' relative humidity recording high - unable to correct.
17 October 75	1.	Differential temperature recording 0.5□F high - sensor would not recalibrate - to be checked at later date.
	2.	250' relative humidity sensor removed from service.
	3.	Drive motor on 30' wind run side marker replaced.
27 December 75	1.	Remove 30' R.H. sensor and circuit board for repair.
	2.	Adjust 30' wind run side marker.

TABLE 2.3-50 (Cont'd)

DATE		FINDINGS
	3.	30' temp. recording 1.25□F high - corrected.
	4.	250' relative humidity not in service.
27 February 76	1.	Delta T recording 1.0□F low - corrected.
	2.	30' R.H. high - unable to correct.
	3.	250' relative humidity not in service.
28 April 76	1.	30' temp. recording 0.4°F low - corrected.
	2.	Delta T recording 1.5°F low - will not calibrate. Card and sensor removed for investigation.
	3.	250' relative humidity not in service.
29 June 76	1.	30' dew point, 250' dew point and sigma cassette data not in service.
	2.	30' dew point sensor damaged by lightning - removed for repair.
	3.	250' dew point not in service.
26 August 76	1.	30' dew point, 250' dew point and sigma cassette data not in service.
	2.	250' delta T recording 0.5°F high - corrected.
	3.	Both dew point sensors not in service.
	4.	Wind recorder lamp out - to be replaced.
	5.	Rain gauge "one shot" inoperative - replaced.
	6.	30' wind direction recorder 2% high - corrected.
3 September 76	1.	Adjust temperature recorder zero by 2□.
	2.	30' dew point, 250' dew point and sigma cassette data not in service.

TABLE 2.3-50 (Cont'd)

DATE	FINDINGS
	3. 30' temperature recording 6.0°F low - corrected.
8 November 76	1. Delta T recording 3.2°F low. Replaced op amps on PCB and recalibrated.
	2. Sigma divider had bad connection which doubled the output values - corrected.

TABLE 2.3-51

METEOROLOGY RESEARCH, INC - METEOROLOGICAL MONITORING PROGRAM

MAINTENANCE LOG FOR MAY 16, 1973 THROUGH APRIL 30, 1975

DATES	HOURS LOST (OVER 50)	INSTRUMENT MEASUREMENT	CAUSE	ACTION TAKEN
May 16, 1973-June 14, 1973	220.74	30' WD, 250' WD 250' WS	Power outage	Power restored
	334.65	250' RH	Sensor not working properly	Repair RH sensor
	664.78	RF	Rain gauge inoperative	Repair rain gauge
June 14, 1973-July 30, 1973	1098.87	250' RH	Faulty sensors	Repair RH sensors
July 30, 1973-Aug. 15, 1973	756.66	30' RH, 250' RH	Faulty sensors	Removed for repairs
Aug. 15, 1973-Sept. 14, 1973	1440.86	30' RH, 250' RH	Out for repair	None
Sept. 14, 1973-Oct. 14, 1973	209.50	30' RH, 250' RH	Out for repair	RH sensor restored
	247.26	All parameters	Lightning strikes	Power restored
	154.00	250' WD	Lightning strikes	Repair circuit board

TABLE 2.3-51 (Cont'd)

DATES	HOURS LOST (OVER 50)	INSTRUMENT MEASUREMENT	CAUSE	ACTION TAKEN
Oct. 26, 1973-Dec. 12, 1973	214.16	30' WD, 250' WD	Ice on sensors	None
	254.78	250' WS	Magnet and shaft malfunction	Repaired
	333.07	RF	Rain gauge damaged in snowstorm	New rain gauge ordered
Dec. 12, 1973-Jan. 21, 1974	1005.40	F	Out of operation	New rain gauge installed
	146.49	30' WD, 30' WS, 250' WD	Ice on sensors	None
Jan. 21, 1974-Feb. 18, 1974	183.40	30' WS, 250' WS	Magnetic gears and cog malfunction	System repaired
	496.13	RF	Rain gauge problem	Replacement and field adjustment
Feb. 18, 1974-March 18, 1974	68.31	All sensors	Thunder and lightning storm	Power restored
	199.17	RF	Rain gauge malfunction	Install heating assembly
	228.18	250' WS	Gear assembly and Reed switches malfunction	Install new gear assembly and Reed switches

TABLE 2.3-51 (Cont'd)

DATES	HOURS LOST (OVER 50)	INSTRUMENT MEASUREMENT	CAUSE	ACTION TAKEN
April 29, 1974- May 28, 1974	--	No major outages		
June 1, 1974- June 30, 1974	50.58	All sensors	Thunderstorm	Power restored
	96.84	30' WD, 250' WD	Thunderstorm	Correct false switching
July 1, 1974- July 31, 1974	247.51	250' RH	Corroded J-box	Repaired in field
	176.53	250' WD	Azimuth board malfunction	Repaired in field
	58.26	30' ΔTemp, 250' ΔTemp	Thunderstorm	Component repaired
Aug. 1, 1974- Aug. 31, 1974	1484.08	30' RH, 250' RH	Sensor malfunction	New sensor ordered
Sept. 1, 1974- Sept. 30, 1974	1440.00	30' RH, 250' RH	Sensor malfunction	None
Oct. 1, 1974- Nov. 11, 1974		Missing data		
Dec. 1, 1974- Dec. 31, 1974	1486.84	30' RH, 250' RH	Sensor malfunction	None
	83.33	30' WD, 250' WD 250' WS	Sensor icing	None

TABLE 2.3-52

MURRAY AND TRETTEL, INC. - METEOROLOGICAL MONITORING PROGRAM

MAINTENANCE LOG FOR MAY 1, 1975 THROUGH NOVEMBER 8, 1976

MAJOR INSTRUMENT OUTAGES

MEASUREMENT	DATES (TIME IN CST)	CAUSE	ACTION TAKEN
30' W.S.	1500 May 14, 1975- 1700 May 20, 1975	Intermittent side marker	Replace side marker
	1900 Oct. 15, 1975- 1000 Oct. 17, 1975	Intermittent side marker	Replace side marker
	2000 Mar. 4, 1976- 1200 Mar. 10, 1976	Lightning	Restore power supply
	2100 Jul. 20, 1976- 1200 Jul. 30, 1976	Lightning	Replace power supply
30' W.D.	2000 Mar. 4, 1976- 1200 Mar. 10, 1976	Lightning	Repair signal conditioning circuit board
	2100 Jul. 20, 1976- 1200 Jul. 30, 1976	Lightning	Replace power supply
30' Temp.	2000 Mar. 4, 1976- 1200 Mar. 10, 1976	Lightning	Restore power supply
	2100 Jul. 20, 1976- 1700 Aug. 4, 1976	Lightning	Replace circuit board with standby unit

TABLE 2.3-52 (Cont'd)

MEASUREMENT	DATES (TIME IN CST)	CAUSE	ACTION TAKEN
250' W.S.	2000 Mar. 4, 1976- 1100 Apr. 9, 1976	Lightning	Replace frozen bearings in sensor
	2100 Jul. 20, 1976- 1200 Jul. 30, 1976	Lightning	Replace power supply
	2100 Dec. 6, 1976- 1000 Dec. 8, 1976	Intermittent sensor failures	Replace reed switch in sensor
250' W.D.	2000 Mar. 4, 1976- 1200 Mar. 17, 1976	Lightning	Replace open sensor pot with spare
	2100 Jul. 20, 1976- 1100 Jul. 30, 1976	Lightning	Replace power supply
250' T	1300 Oct. 17, 1975- 1000 Oct. 21, 1975	Corrosion in J-box	Replace terminal strip in ground level J-box
	2000 Mar. 4, 1976- 1200 Mar. 10, 1976	Lightning	Repair signal conditioning circuit board
	2100 Jul. 20, 1976 1700 Aug. 4, 1976	Lightning	Install and calibrate new PCBs
	0100 Oct. 28, 1976- 1000 Nov. 8, 1976	Low values being recorded	Replace op amps and recalibrate system

TABLE 2.3-53

BYRON DATA RECOVERY (%)

INSTRUMENT MEASUREMENT	1974	1975	1976
30' wind speed	94.2	92.4	95.0
30' wind direction	98.2	94.1	93.0
30' temperature	99.5	98.9	94.6
30' RH/dew point	51.4	65.2*	28.7**
250' wind speed	96.2	94.9	85.2
250' wind direction	98.8	92.6	89.9
250' delta T	99.2	93.5	88.9
250' RH/dew point	50.4	69.8*	14.6***
Precip.	87.1	96.7	94.8
30' ws, wd, 250ΔT	91.8	81.3	78.5
250' ws, wd, ΔT	94.3	82.2	68.1

*MRI relative humidity sensors

**30-foot CTE dew point sensor installed June 16, 1976. 306 hours of data were recovered before the system failed due to a severe lightning strike. The system was reinstalled September 16, 1976.

***250-foot CTE dew point sensor installed November 8, 1976.

TABLE 2.3-54

MINIMUM EXCLUSION AREA BOUNDARY DISTANCES FOR BYRON

SECTOR	MINIMUM EXCLUSION AREA BOUNDARY DISTANCE (FEET)
N	2900
NNE	1945
NE	1490
ENE	1375
E	1375
ESE	1375
SE	1490
SSE	1945
S	3100
SSW	3100
SW	3355
WSW	3585
W	3900
WNW	3900
NW	3140
NNW	2900

TABLE 2.3-55

BYRON STATION JOINT WIND-STABILITY CLASS FREQUENCY DISTRIBUTION (1994-1998)

30 FT METEOROLOGICAL TOWER LEVEL

		Wind Direction Category																
Stability Class	Wind Speed Category(1) (2)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
1 (A)	2	1	2	0	2	2	3	3	1	1	4	2	6	1	1	3	1	33
	3	28	19	13	13	24	13	8	21	11	15	27	15	20	23	19	18	287
	4	54	47	20	21	36	23	20	43	25	38	54	41	43	64	97	43	669
	5	18	12	4	21	3	7	11	31	38	44	41	32	37	59	51	23	432
	6	0	2	0	5	0	0	0	3	11	13	6	1	10	4	2	1	58
	7	0	0	0	0	0	0	0	1	0	3	1	1	0	0	0	0	6
2 (B)	2	3	5	0	3	5	3	2	2	2	1	0	1	1	2	3	4	37
	3	32	19	7	11	23	23	15	14	14	26	31	14	18	16	26	23	312
	4	64	48	17	11	34	25	41	37	30	48	69	42	52	59	90	44	711
	5	12	5	10	18	2	6	8	16	28	37	21	16	29	40	31	17	296
	6	4	1	0	3	0	1	4	7	6	6	4	6	10	9	3	1	65
	7	1	0	0	0	0	0	0	0	2	2	2	2	0	0	0	0	9
3 (C)	2	4	2	1	1	5	2	2	1	2	2	10	2	3	3	4	5	49
	3	36	29	27	19	66	36	49	25	41	53	47	33	42	46	64	45	658
	4	70	46	30	25	36	26	49	67	72	84	103	53	69	93	121	67	1011
	5	22	13	9	15	4	4	6	18	44	45	33	33	48	68	46	22	430
	6	3	2	1	1	0	1	5	3	5	7	3	8	10	14	4	1	68
	7	1	0	0	0	0	0	0	3	0	1	4	2	1	1	0	0	13
4 (D)	2	50	65	68	62	81	62	56	40	43	49	70	79	72	96	84	80	1057
	3	467	305	267	247	473	269	261	295	362	348	419	360	414	527	573	463	6050
	4	733	413	307	531	366	241	306	486	496	410	495	457	660	788	692	561	7942
	5	304	263	296	322	62	112	121	277	297	319	235	272	534	487	238	165	4304
	6	38	33	78	31	1	30	23	57	49	69	38	86	162	130	15	15	855
	7	10	1	5	0	0	2	0	12	1	9	19	30	62	17	0	0	168
5 (E)	2	80	61	61	70	131	67	71	69	88	106	138	119	166	168	178	101	1674
	3	243	206	141	215	538	290	300	345	416	351	376	333	405	381	418	330	5288
	4	130	87	138	241	146	160	269	442	474	350	295	221	317	161	142	93	3666
	5	10	8	24	61	11	58	42	166	203	225	72	36	60	14	5	6	1001
	6	0	0	0	0	0	16	8	16	17	43	6	2	7	1	0	0	116
	7	0	0	0	0	0	4	0	4	0	0	0	0	0	3	0	0	11
6 (F)	2	55	33	25	20	69	48	40	63	87	103	112	123	183	167	221	103	1452
	3	69	55	26	46	230	254	297	410	379	201	96	69	102	43	81	106	2464
	4	6	3	1	23	11	87	87	228	118	29	3	6	3	1	0	0	606
	5	0	0	0	1	0	1	0	6	3	1	0	0	0	0	0	0	12
	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7 (G)	2	32	12	10	23	46	40	25	50	79	85	77	82	112	117	188	116	1094
	3	14	2	1	5	67	96	93	95	157	41	7	6	11	7	13	10	625
	4	1	0	0	5	5	20	21	30	16	0	0	0	0	0	0	0	98
	5	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	1
	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Notes:
1) Wind speed categories defined as follows:

Category	Wind Speed (mph)
2	≥0.8 to <3.5
3	≥3.5 to <7.5
4	≥7.5 to <12.5
5	≥12.5 to <18.5
6	≥18.5 to <24
7	≥24

2) Wind speed Category 1 is assumed for calm occurrences.
Calm occurrences by stability class: A=0, B=0, C=0, D=4, E=7, F=5, G=11

TABLE 2.3-56

BYRON STATION JOINT WIND-STABIILTY CLASS FREQUENCY DISTRIBUTION (1994-1998)

250 FT METEOROLOGICAL TOWER LEVEL

		Wind Direction Category																
Stability Class	Wind Speed Category[1] [2]	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
1 (A)	2	1	1	2	0	2	1	2	1	1	0	2	4	1	0	0	3	21
	3	4	5	4	7	7	5	6	6	10	7	25	14	10	8	12	7	137
	4	55	39	19	11	41	19	15	32	19	29	30	21	22	42	36	31	461
	5	29	26	14	14	22	12	15	28	26	44	47	31	46	51	95	37	537
	6	5	4	0	13	2	8	7	19	20	19	20	20	19	38	38	9	241
	7	0	2	0	5	2	2	2	6	19	15	10	4	10	6	4	1	88
2 (B)	2	2	1	2	1	2	3	3	2	1	0	0	1	2	1	1	1	23
	3	15	10	6	7	15	12	5	10	4	8	18	7	9	6	11	18	161
	4	44	34	16	8	30	20	31	31	28	40	45	32	34	36	47	30	506
	5	34	27	11	6	18	18	21	20	32	42	42	27	37	55	68	34	492
	6	4	3	8	15	2	3	5	9	14	13	11	10	22	22	14	10	165
	7	4	1	0	4	0	4	4	5	11	8	6	8	11	8	7	2	83
3 (C)	2	2	1	1	0	0	0	1	0	1	2	3	1	1	1	2	3	19
	3	27	10	21	15	34	37	30	19	24	31	33	19	23	19	28	23	393
	4	52	46	17	15	51	20	33	42	65	65	68	47	48	81	85	49	784
	5	30	25	21	18	18	14	29	43	40	53	66	38	50	90	90	53	678
	6	11	8	6	7	4	6	7	11	23	25	18	21	23	36	25	10	241
	7	3	2	2	2	0	2	5	7	10	9	12	12	10	18	8	3	105
4 (D)	2	31	26	31	35	33	20	23	15	29	27	22	28	32	41	29	29	451
	3	214	167	142	156	227	145	135	127	150	173	191	191	216	233	249	235	2951
	4	424	349	213	244	385	219	178	265	311	324	354	369	432	486	552	451	5556
	5	532	405	218	359	372	180	192	319	354	334	384	413	611	697	589	418	6377
	6	199	180	193	212	128	102	144	192	220	250	216	178	336	334	178	113	3175
	7	59	92	146	94	26	67	90	194	137	133	123	120	258	184	50	20	1793
5 (E)	2	11	17	31	20	28	13	9	8	10	11	16	13	15	20	23	15	260
	3	76	63	62	78	147	61	55	47	40	51	60	64	64	64	72	88	1092
	4	159	107	133	141	330	130	116	102	153	179	228	222	247	323	339	222	3131
	5	194	185	133	203	244	189	219	232	339	429	416	338	465	342	351	229	4508
	6	24	24	33	92	58	129	167	243	313	400	225	91	87	43	28	19	1976
	7	0	2	2	7	2	75	61	166	162	163	64	6	19	6	1	1	737
6 (F)	2	5	5	7	12	13	9	4	8	2	3	5	10	6	8	4	12	113
	3	36	27	31	33	39	24	24	28	31	25	23	25	31	33	32	35	477
	4	78	50	39	30	100	50	56	68	86	77	79	72	89	140	120	98	1232
	5	71	63	46	21	56	113	174	116	225	216	197	104	91	103	101	105	1802
	6	3	6	1	4	10	93	117	161	205	143	28	10	3	1	1	1	787
	7	0	0	0	0	0	10	13	38	43	5	0	1	0	0	0	0	110
7 (G)	2	11	11	5	6	6	5	4	1	8	2	8	8	10	7	5	12	109
	3	20	22	38	40	29	11	9	6	9	21	13	10	21	23	35	32	339
	4	49	30	27	12	22	22	31	28	48	28	36	26	44	32	42	75	552
	5	27	26	10	1	7	37	71	68	66	65	65	28	33	25	53	30	612
	6	0	1	0	1	4	34	44	38	35	25	9	0	1	0	0	0	192
	7	0	0	0	0	0	4	8	10	3	0	0	0	0	0	0	0	25

Notes:

1) Wind speed categories defined as follows:

Category	Wind Speed (mph)
2	≥0.8 to <3.5
3	≥3.5 to <7.5
4	≥7.5 to <12.5
5	≥12.5 to <18.5
6	≥18.5 to <24
7	≥24

2) Wind speed Category 1 is assumed for calm occurrences.
Calm occurrences by stability class: A=0, B=0, C=0, D=2, E=2, F=1, G=1

TABLE 2.3-57

ARCON96 INPUT PARAMETER SUMMARY FOR BYRON STATION

	ARCON96 INPUT PARAMETER	Control Room Fresh Air Intake: Containment Wall	Control Room Fresh Air Intake: Plant Vent	Control Room Fresh Air Intake: PORVs/ Safety Valves	Control Room Fresh Air Intake: Main Steam Line Break	Turbine Building Emergency Air Intake: Containment Wall	Turbine Building Emergency Air Intake: Plant Vent	Turbine Building Emergency Air Intake: PORVs/ Safety Valves	Turbine Building Emergency Air Intake: Main Steam Line Break
UNIT 1	Release Height (m)	29.7	61	9.8	7.9	29.7	61	9.8	7.9
	Intake Height (m)	21.2	21.2	21.2	7.9	20.4	20.4	20.4	20.4
	Horizontal Distance from Intake to Stack (m)	7.6	34.1	22.9	43.3	30.5	27.4	35.1	13.4
	Elevation Difference between Stack Grade and Intake Grade (m)	0	0	0	0	0	0	0	0
	Building Area (m^2)	2916.7	2227.6	2916.7	2850.7	2916.7	752.6	2916.7	752.6
	Direction from Intake To Stack (°)	255	37	192	60	262	356	231	356
	Vertical Velocity (m/s)	0	0	0	0	0	0	0	0
	Stack Flow (m^3/s)	0	0	0	0	0	0	0	0
	Stack Radius (m)	0	0	0	0	0	0	0	0
	Initial Value of σ_y (m)	8.18	0	0	0	8.18	0	0	0
	Initial Value of σ_z (m)	9.9	0	0	0	9.9	0	0	0
	Minimum Wind Speed (m/s)	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
	Surface Roughness Length (m)	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
	Sector Averaging Constant	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3
UNIT 2	Release Height (m)	29.7	61	9.8	7.9	29.7	61	9.8	7.9
	Intake Height (m)	21.2	21.2	21.2	7.9	20.4	20.4	20.4	20.4
	Horizontal Distance from Intake to Stack (m)	7.6	34.1	22.9	43.3	30.5	27.4	35.1	13.4
	Elevation Difference between Stack Grade and Intake Grade (m)	0	0	0	0	0	0	0	0
	Building Area (m^2)	2916.7	2227.6	2916.7	2850.7	2916.7	752.6	2916.7	752.6
	Direction from Intake To Stack (°)	286	143	348	120	279	184	309	184
	Vertical Velocity (m/s)	0	0	0	0	0	0	0	0
	Stack Flow (m^3/s)	0	0	0	0	0	0	0	0
	Stack Radius (m)	0	0	0	0	0	0	0	0
	Initial value of σ_y (m)	8.18	0	0	0	8.18	0	0	0
	Initial value of σ_z (m)	9.9	0	0	0	9.9	0	0	0
	Minimum Wind Speed (m/s)	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
	Surface Roughness Length (m)	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
	Sector Averaging Constant	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3

TABLE 2.3-58

ARCON96 CONTROL ROOM INTAKE χ/Q RESULTS* (sec/m^3)
FOR BYRON STATION

		Control Room Fresh Air Intake				Turbine Building Emergency Air Intake			
		Containment Wall	Plant Vent	PORVs/ Safety Valves**	Main Steam Line Break	Containment Wall	Plant Vent	PORVs/ Safety Valves**	Main Steam Line Break
UNIT 1	0-2 hour	1.36E-03	1.95E-03	**1.77E-03**	2.87E-03	9.06E-04	**2.29E-03**	**8.10E-04**	1.49E-02
	2-8 hour	8.51E-04	1.42E-03	**1.52E-03**	2.37E-03	5.34E-04	**1.92E-03**	**6.70E-04**	1.21E-02
	8-24 hour	3.46E-04	**6.13E-04**	**6.98E-04**	1.05E-03	2.29E-04	7.53E-04	2.70E-04	4.83E-03
	1-4 days	2.46E-04	3.77E-04	**4.72E-04**	6.13E-04	1.63E-04	4.85E-04	**1.95E-04**	2.96E-03
	4-30 days	1.95E-04	2.59E-04	**3.50E-04**	**4.61E-04**	1.24E-04	3.49E-04	1.35E-04	2.10E-03
UNIT 2	0-2 hour	**1.44E-03**	**1.99E-03**	1.57E-03	**3.20E-03**	**9.32E-04**	2.28E-03	7.84E-04	**1.70E-02**
	2-8 hour	**9.76E-04**	**1.46E-03**	1.29E-03	**2.57E-03**	**5.85E-04**	1.86E-03	6.98E-04	**1.46E-02**
	8-24 hour	**4.08E-04**	6.07E-04	5.28E-04	**1.07E-03**	**2.49E-04**	**8.14E-04**	**3.04E-04**	**6.68E-03**
	1-4 days	**2.88E-04**	**3.79E-04**	3.28E-04	**6.17E-04**	**1.76E-04**	**5.52E-04**	1.89E-04	**4.48E-03**
	4-30 days	**2.12E-04**	**2.87E-04**	2.34E-04	**4.61E-04**	**1.29E-04**	**4.40E-04**	**1.67E-04**	**3.31E-03**

* Bolding and shading indicates the maximum unit χ/Q value.
** PORVs/Safety Valve χ/Q values contain a factor of 5 reduction for vertical uncapped releases per RG 1.194, Section 6.

2.4 HYDROLOGIC ENGINEERING

2.4.1 Hydrologic Description

2.4.1.1 Site and Facilities

Byron Station is located 3 miles southwest of Byron in Ogle County, in north central Illinois, and 2 miles east of the Rock River at about river mile 115 from the confluence with the Mississippi River. The plant site occupies about 1300 acres and includes a portion of Woodland Creek which is an intermittent stream and a 3-mile long tributary to the Rock River. A topographic map of the site and its vicinity is given in Figure 2.4-1.

The elevations of the Rock River at the site corresponding to the mean annual flow and the probable maximum flood (PMF) are 672.0 feet and 708.3 feet, respectively (all elevations refer to USGS 1929 datum). The plant grade elevation is 869.0 feet.

The river screen house, which withdraws makeup water from the Rock River, is protected from the combined event flood with coincident wind wave activity (see Subsection 2.4.3.9). The safety-related equipment at the river screen house is at elevation 702.0 feet. Figure 2.4-2 shows the river screen house and makeup lines. All other Seismic Category I structures, such as the reactor containment buildings, auxiliary building, fuel handling building, and essential service water cooling towers are at or above elevation 869 feet, i.e., 161 feet or more above the PMF level of the Rock River. The deep wells are also above the PMF level of the Rock River. The outline of all major structures is shown in Figure 2.4-3.

Access to the plant site would be maintained during any flood conditions. The site is bounded by County Highway 2 (German Church Road), Deerpath Road, and Razorville Road. The road elevations vary from 769 feet to 883 feet. None of the above roads would be affected by the PMF from the Rock River. Other principal roads in the area are Illinois State Highway 72, about 3 miles northeast and Illinois Highway 64, about 4 miles south. The plant site is also accessible from the Chicago and North Western Railroad which is about 3 miles northeast of the site. Ground topography along the railroad tracks is high and the tracks are well above the PMF level of the Rock River.

2.4.1.2 Hydrosphere

The Rock River rises in Fond du Lac County in southeastern Wisconsin and flows in a southerly direction into Illinois. The headwater area contains many natural lakes which control both flood and low flows. In Illinois the river changes to a southwesterly direction, entering the Mississippi River downstream from Rock Island, Illinois. Of the Rock River drainage area, 5574 mi^2 are in Wisconsin and 5343 mi^2 are in Illinois

(Reference 1). The drainage area upstream of the site is about 8170 mi^2. Table 2.4-1 gives drainage areas of the Rock River at various locations.

There are two USGS river gauging stations on the Rock River near the site. The gauging station at Rockton, 41 miles upstream of the site, has flow measurements for the following periods: June 1903 to July 1906, October 1906 to March 1909, July 1914 to September 1919, and October 1939 to current year. The Como gauging station, 45 miles downstream of the site, has the following flow data: March to December 1905, and October 1914 to September 1971, and from 1977 to the current year. From 1972 through 1977, only annual maximum discharges are available at Como. The hydrologic network of the Rock River and its tributaries is shown in Figure 2.4-4.

Near the site location, the Rock River is partially regulated by low dams which are located at Rockton (11 feet high), 44 miles upstream from the plant river screen house; Rockford (13 feet), 22 miles upstream; Oregon (10 feet), 5 miles downstream; Dixon (12.4 feet), 27 miles downstream; and two at Sterling (8.9 and 8.8 feet), 41 and 42 miles downstream. These dam locations are shown on Figure 2.4-5. Table 2.4-2 lists dams on the Rock River near the site.

The Rockford, Dixon, and lower Sterling dams are made of wood cribbing filled with rock and sand and capped with concrete. The structures are founded on steel pilings. The upper Sterling dam comprises 570 feet of concrete sluiceways, steel gates, and wooden sills, and 590 feet of wood cribbing filled with rock, concrete-filled steel pilings, concrete apron, and large boulders on the upstream side.

Public and private Rock River surface water users are shown on Figure 2.4-5. The Rock River and its tributaries are not used for public water supply in Illinois or for navigation. There are a few private users of water between Wisconsin state line and about 50 miles downstream from the station. There is one farmer who consumes less than 0.5 cfs of Rock River for irrigation during the summer. He is located near Prophetstown, which is about 55 miles downstream. There are no other known irrigation uses in Lee, Ogle, Boone, Winnebago, or Whiteside counties. Table 2.4-3 gives the owner and location of surface water intakes for industrial use downstream of the site. Medusa Cement in Dixon has an average withdrawal of 0.25 cfs. Northwestern Steel and Wire in Sterling withdraws an average of 41 cfs, a peak of 48 cfs, a minimum of 33 cfs, and consumes an average of 2 cfs. The rest of the diverted water is returned to the river. The steam generating station in Dixon is no longer in operation. The hydroelectric plant at Dixon does not operate during floods; at other times the maximum flow through the plant is 5000 cfs. About 70 cfs is diverted near Sterling from the river into the Illinois and Mississippi canal.

Subsection 2.4.13 lists groundwater users in the vicinity of the Byron Station.

2.4.2 Floods

2.4.2.1 Flood History

Table 2.4-4 includes all recorded floods through September 1976, which produced a stage of greater than 10.0 feet (elevation 717.94) at the USGS gauging station located on the left bank of the Rock River at Rockton (References 2 and 3). The gauge is located 41 miles upstream of the station withdrawal point and intercepts 78% of the drainage area at the plant intake. Present gauge datum is at elevation 707.94 feet. Gauge datum was about a foot higher from June 1903 to September 1906, and about 2 feet higher from October 1906 to March 1909. Gauge heights listed for the period 1915 to 1919 are from the gauge at Rockford (gauge datum unknown).

The flood of March 1975 reached the maximum stage of 15.54 feet at Rockton. The flood with the highest recorded discharge, 32,500 cfs, occurred March 30, 1916 at Rockton, reaching a stage of 13.06 feet.

The flood of record at the Como gauging station was 59,700 cfs on April 22, 1973. The estimated flood of record at the site area was 57,700 cfs on April 22, 1973. See Subsection 2.4.2.2 for an explanation of the formula used to interpolate flows near the project area.

The mean annual flow at Rockton is 3901 cfs; at Como, 4996 cfs; and at the site area, 4728 cfs. The rating curves for the Como and Rockton gauges are shown on Figures 2.4-6 and 2.4-7, respectively. Rating curves at these two stations are given for information.

Table 2.4-5 lists the ten largest floods for the Rock River at Rockton through September 1976 in order of flow magnitude. Datum levels are the same as explained for Table 2.4-4.

Flood duration on the Rock River is relatively long. During the April 1959 flood, for example, the water rose from an initial stage of 4.2 feet at Rockton to a peak stage of 14.18 feet in 16 days and remained above the flood stage of 10.0 feet for 21 days. During the March 1948 flood, the river rose to a peak stage of 13.8 feet in 6 days and remained above the flood stage for 11 days.

2.4.2.2 Flood Design Considerations

Since there is no gauging station located at the site, it is necessary to develop a correlation between flows at the site and those at the nearby gauging stations. The drainage area between the two gauging stations is 2392 mi^2. The area between the Byron site and the Rockton gauging station is 1807 mi^2 or 75.5% of the drainage area between Rockton and Como gauging stations. The area between the Como gauging station and the site is 585 mi^2 or 24.5% of the drainage area between Rockton and Como gauging stations. For low or normal flows, the flow at the Byron site may be expressed as the sum of weighted flows at Rockton and Como. This approach gives:

$$Q_{site} = 0.245\ Q_{Rockton} + 0.755\ Q_{Como} \qquad (2.4\text{-}1)$$

For the purposes of predicting extremely high flows, the flood data from the Como gauging station is more reliable and is appropriate. This conclusion is reached because of the longer uninterrupted flood record at Como and because the flood flows at Como are more representative of the flows at the Byron site. Also the drainage area at Byron is 93% of that at Como. The annual maximum flood flow at the Byron site is calculated, using the following equation:

$$Q_{site} = 0.966\ Q_{Como} \qquad (2.4\text{-}2)$$

The coefficient of 0.966 is obtained by taking the square root of the ratio of the drainage areas. The drainage area at Como station is 8755 mi^2 and at the Byron site is 8170 mi^2.

A Log-Pearson Type III frequency analysis of the maximum flood flows at the Byron site from 1915 to 1974 was made and the calculated discharges for five recurrence intervals are given in Table 2.4-6. By including the data of 1975 and 1976 in the above frequency analysis, the change in peak discharge for any given recurrence interval is insignificant. Further details of the analysis are given in Subsection 2.4.3.7.

The river screen house is the only structure that could be affected by flooding on the Rock River and is designed for the combined event flood (defined in Subsection 2.4.3.7). The design bases for the river screen house, under both high and low water conditions, are discussed in Subsections 2.4.3 and 2.4.11, respectively. All other structures are 161 feet or more above the PMF level of the Rock River.

Landslides and dam failures can cause or increase floods in two ways: (1) by creating a surge on large rivers, lakes, and reservoirs and (2) by blocking a river and then releasing the stored water when the blockage is overtopped and washed out. The effect of a surge decreases very rapidly in a downstream direction. Since the site area is remote from upstream gorge-type topography and the nearest dam is 22 miles upstream, an increase in elevation of the Rock River near the site from such a surge would be negligible. Also, since there is no gorge downstream, no landslide could occur that would cause an appreciable increase in river level at the site area due to water backing up.

2.4.2.3 Effects of Local Intense Precipitation

Grading and drainage at the Byron site are designed to ensure that no flooding of safety-related facilities will occur for precipitation events up to the severity of the probable maximum precipitation (PMP). The layout of roads, tracks, and drainage in the immediate plant area is shown in Figure 2.4-8.

The 48-hour all-season PMP is estimated to be 34.28 inches from Hydrometeorological Report No. 33 (Reference 4). The all-season PMP is chosen since it is greater than the winter PMP combined with the 100-year snowpack. The critical rainfall distribution based on the U.S. Army Corps of Engineers Memorandum 1110-2-1411 is given in Table 2.4-7.

The roofs of all safety-related structures are designed to withstand the higher of the loads caused by the 24-hour all season PMP or the 100-year maximum snow pack combined with the winter PMP of 48-hour duration at the plant site.

Postulating that the roof drains get clogged at the time of PMP, the maximum accumulation of water on the roofs of safety-related structures will be up to the height of the parapet walls plus the depth of overflow over the parapet wall. The height of the parapet walls is 1 foot 4 inches. The maximum depth of overflow is estimated to be 2 inches. Therefore, the corresponding water load due to summer PMP on the roofs will be 93.6 lb/ft^2. The maximum 48-hour winter PMP at the site is 14.7 inches in March (Reference 4). The snow load at the site corresponding to a 100-year mean recurrence interval is 28 lb/ft^2. Due to the 1-foot, 4-inch-high parapet walls, accumulation of the entire winter PMP on the roofs with the above snow load is not possible, and the excess precipitation overflows the parapet. Therefore, the governing roof load is 93.6 lb/ft. However, as explained in Subsection 3.8.4, the roofs of all safety-related structures are designed for a load of 104 lb/ft^2.

The plant grade elevation is 869.0 feet and the grade floors of the safety-related buildings are at elevation 870.0 feet.

The site drainage system is designed to follow the natural drainage pattern and to drain the storm water away from the plant area by means of ditches and culverts. Pertinent data for culverts are shown in Table 2.4-7b. The areas surrounding the plant are graded to direct the surface runoff to the existing natural streams, north and west of the plant area.

For the analysis of local intense precipitation, the 1-hour PMP on 1-mi^2 area for the site is taken from the Hydrometeorological Report No. 52 (Reference 4b). This PMP was derived from

6-hour, 10-mi^2 PMP values given in Hydrometeorological Report No. 51 (Reference 4a) and is considered the point rainfall value. This 1-hour, 1-mi^2 PMP is distributed into values for small durations, following procedures given in Reference 4b. This magnitude and intensity of these smaller duration rainfalls are presented in Table 2.4-7a.

The runoff from local intense precipitation that can contribute to potential flooding of the plant area is from the areas shown in Figure 2.4-9. The probable maximum precipitation falling on this area was considered in the analysis of local intense precipitation on the plant site. The peripheral roads and railroads will act as weirs in the event of a PMP at the plant site, to pass the resulting runoff. In addition, the precast concrete barriers along the outside security fence are considered in the analysis. The barrier placement would cause a restricted flowpath for the PMP runoff. The barriers are placed along the edge of the existing roads or the parking areas outside the security fence. The elevation of the grade on which the barriers are placed is considered in the analysis, and the water level upstream of the barriers is calculated assuming weir flow through the openings. The calculated water level upstream of the barriers is considered as the tail water level for the weir flow over the peripheral plant roads and railroad tracks inside the security fence to calculate the maximum water level near the plant buildings. Cable-type barriers are used in place of the concrete barriers wherever the concrete barriers severely constrict the flow path.

Furthermore, the concrete barriers used as part of security upgrades are considered in the analysis. These barriers are placed on grade to the west and south of the reactor buildings. The openings between the concrete barriers are considered in the analysis as a broad crested weir to calculate the maximum water level upstream of the barriers.

It was conservatively assumed in the analysis that the site drainage system would not be functioning at the time of the PMP. The rational formula was used in estimating the peak runoff from the area. The rational formula is given by:

$$Q = CIA,$$

where:

Q = peak runoff (cfs),

C = coefficient of runoff,

I = intensity of rainfall (inches/hour), and

A = drainage area (acres).

The coefficient of runoff was assumed conservatively to be 1.0. The time of concentration was computed from Kirpich's formula (Reference 34). The intensity of rainfall corresponding to a time of concentration was interpolated from Table 2.4-7a. The water surface elevation was estimated for peak flow over the peripheral roads and railroad track through the openings between the peripheral concrete barriers, and through the openings between the security upgrades concrete barriers using a broad-crested weir formula with a coefficient of discharge of 2.64 (Reference 4c). Whenever the tail water level is higher than the weir crest, submergence factors are applied to the coefficient of discharge. Backwater calculations are performed for the areas from the peripheral roads and railroad tracks upstream to plant buildings to estimate the maximum water level adjacent to the buildings housing the safety-related equipment.

The plant site area is divided into Zones A, B, C, D and E as shown in Figure 2.4-9.

The stormwater that accumulates in Zone A flows through the openings in the peripheral concrete barriers north, west, and south of the zone. The runoff from the inner area bounded by the railroad tracks and plant roads inside the security fence in Zone A flows over railroad Track 1 on the west, over railroad Track 3 on the north, and over the plant road on the south.

The total area of Zone A is 35.11 acres. The time of concentration is 19.4 minutes, with a peak runoff from Zone A of 1159 cfs. Due to the maximum water level in Zone B being higher than the maximum water level estimated for Zone A, the spillover flow from Zone B into Zone A was considered. Therefore, the revised runoff from Zone A during a PMP is 1459 cfs. This peak flow produces a water level of 869.4 feet upstream of the concrete barriers. The revised peak runoff from the inner area of 26.49 acres in Zone A is 1175 cfs. The maximum water level near the plant buildings due to peak flow over 1160 feet of track at elevation 870.17 feet is 870.82 feet, including the effect of backwater.

The area of Zone B is 41.7 acres, and with a time of concentration of 20 minutes, the peak runoff from Zone B is 1355 cfs. Considering a spillover flow from Zone B to Zone A, as discussed earlier, the revised peak runoff from Zone B is 1055 cfs. This peak flow overflows 235 feet of roads at the north boundary of Zone B and produces a maximum water level of 870.90 feet near the plant buildings, including the effects of backwater. The effect of submergence at the outflow from Zone B due to water level in Zone D is considered in the analysis.

Zone C has an area of 22 acres, and the runoff from the zone flows east and north of the cooling towers into the natural stream. The peak flow from Zone C is 814 cfs, and the road east of the zone slopes from elevation 870.0 feet at the middle of the zone to elevation 857.0 feet at the northeast corner of the north cooling tower. Further, the roads around the cooling tower are at an elevation of 873.0 feet. Therefore, the peak runoff from Zone C will flow away and will not cause any flooding near the safety-related facilities of the station.

The runoff from Zone D flows over the road along the east boundary of the area and over the peripheral railroad track on the west and north. The drainage area of Zone D is 22.62 acres, and the peak runoff from the area is 667 cfs, with a time of concentration of 24 minutes. This peak flow over the east road at elevations 869.25 feet and 870.0 feet, and over the railroad track at elevation 870.17, produces a maximum water level of 870.3 feet near the south boundary of Zone D.

Zone E has an area of 15.6 acres, and the runoff from this zone flows over the railroad Track 1 on the west. The time of concentration for the flow in this area is 18 minutes, and the peak runoff is 538 cfs. This peak runoff flows over the Track 1, 750 feet long at elevation 870.17 feet, and produces a maximum water level of 870.59 feet in Zone E.

It can be seen from the foregoing analysis that the water level in the area adjacent to the plant buildings can reach an elevation above the grade floor level of 870.0 feet. This occurs for a short period of time until the PMP is discharged from the plant site by overflow over the roads and railroads and the drainage system. To prevent this water from entering areas where essential equipment/systems are located, reinforced concrete curbs or steel barriers are provided.

2.4.3 Probable Maximum Flood (PMF) on the Rock River

The probable maximum flood is defined by the Corps of Engineers as the hypothetical flood characteristics that are considered to be the more severe reasonably possible at a particular location, based on relatively comprehensive hydrometeorological analysis of critical runoff-producing precipitation and hydrologic factors favorable for maximum flood runoff.

The peak discharge of the probable maximum flood for the Rock River at the intake point was calculated to be 308,000 cfs. This discharge was derived by doubling the peak discharge of the standard project flood as described in the following sections.

Flooding in Woodland Creek is insignificant since the site intercepts less than 1 mi^2 of the drainage area of the creek.

2.4.3.1 Standard Project Flood and Other Floods on the Rock River

The standard project flood (SPF) is the flood expected from the most severe combination of meteorological and hydrologic conditions that are considered reasonably characteristic of the geographical region involved, excluding extremely rare combinations. It is the design flood for a project where some small degree of risk can be accepted but where a high degree of protection is justified. Though the SPF could occur in any given year, it would be an extremely rare event and is not assigned a frequency of occurrence. The SPF for the Rock River near the site was estimated to have a peak of 154,000 cfs on the basis of a synthetic storm which has a depth equal to half of the probable maximum precipitation.

2.4.3.2 Standard Project Storm

Depth-duration curves for PMP, one-half PMP, and two major storms which have occurred in the vicinity of the project area are shown on Figure 2.4-10. The two historical storms shown on the figure are the Upper Mississippi Valley (UMV) 1-22 storm of August 1941 which centered near Minong, Wisconsin, and the July 1957 storm centered near Kankakee, Illinois. All of the depth-duration curves shown in Figure 2.4-10 correspond to the average over the drainage area of the Rock River above the project site. Routing studies showed that the peak magnitude of any large flood is much more strongly influenced by the maximum 6-hour precipitation than total storm precipitation. As indicated in the figure, the two historical storms had a maximum 6-hour precipitation of 2.6 inches, or 37% of the 6-hour PMP. Fifty percent of the latter value, or 3.5 inches in 6 hours, is adequately conservative in representing rainfall in the region for all but extremely rare storms.

A synthetic standard project storm (SPS) was constructed on the basis of the method described in the U.S. Army Corps of Engineers publication EM-1110-2-1411, titled, "Standard Project Flood Determination," using a depth equal to one-half Of the probable maximum precipitation given in the Hydrometeorological Report No. 33 (HM33) and a draft revision of the report which is applicable to areas up to 20,000 mi^2. The storm was superimposed on the Rock River basin upstream from the project site as shown in Figure 2.4-11. The storm was located and oriented to produce the most critical flood near the project site. The average depth of precipitation for each sub-basin was calculated, and its time distribution was determined as shown in Table 2.4-8. The time distribution for each sub-basin was determined by a two-step procedure:

a. estimate incremental precipitation per unit time for the sub-basin hydrograph according to depth-area duration relationship given in HM33 and its draft revision, and

b. arrange increments of precipitation in sequences which yield the most critical flood.

2.4.3.3 Precipitation Losses

Part of a storm's rainfall is retained by the basin and does not contribute to direct runoff. The significant factors in basin retention are depression storage, interception by vegetation, and concentration. The rate of retention depends largely on soil land use, and antecedent soil moisture conditions.

Initial precipitation retention of 1 inch followed by uniform hourly retention rates based on soil and land use were used in deriving the SPF. Soils of the Rock River drainage area vary from well drained to somewhat poorly drained inorganic soil or organic soils overlying calcareous outwash, till, or lacustrine deposits. Based on county soils maps, and soils information obtained directly from Soil Conservation Service agents, hydrologic soil groups were determined from each sub-basin. Table 2.4-9 shows the soil groups and minimum retention rate for each sub-basin.

2.4.3.4 Runoff Model

Unit hydrographs for sub-basins of the Rock River drainage area have been derived by the U.S. Army Corps of Engineers (Reference 5). The unit hydrograph for the Kishwaukee River at Perryville (Sub-basin III) has been adopted in this study without modification. Unit hydrographs for the Rock River above Rockton and the Pecatonica River at its mouth are also available from the U.S. Army Corps of Engineers, but they were not used directly in this study. Overlay of the SPS isohyetals on the sub-basin map shows significant variations of precipitation over these large sub-basins. The large variations do not conform to the basic assumption of the unit hydrograph theory that precipitation is uniform over each sub-basin. The sub-basins were, therefore, divided into smaller sub-basins to account for a real variation of the SPS. Unit hydrographs were derived for each sub-basin as shown on Figures 2.4-12, 2.4-13, and 2.4-14. The graph for Sub-basin VI was adopted directly from that published in "Unit Hydrographs in Illinois" (Reference 6). Snyder's method was used to derive the unit hydrographs. The coefficients C_t and C_p were based on those determined by the U.S. Army Corps of Engineers from recorded floods for the same general area. However, the peak of the unit hydrograph for Sub-basin I, at the upper end of the watershed, was increased by 25%. This adjustment was made to account for the relatively large routing effect of the lakes and marshes in the sub-basin. The natural storm effect should not be as significant during the SPF as it likely was for the recorded flood used by the Corps to derive their unit hydrograph. The lag time of the unit hydrograph for Sub-basin I also was reduced by about 25% to account for the reduced significance of storage during the larger storm event.

Parameters for the unit hydrographs and their corresponding sub-basin characteristics are summarized in Table 2.4-10.

2.4.3.5 Standard Project Flood Flow

Based on the SPS and retention rates given in Tables 2.4-8 and 2.4-9 rainfall excess for each sub-basin was determined and applied to the corresponding unit hydrograph to obtain the SPF for the sub-basin. The flood hydrographs for the sub-basins were combined and routed downstream, according to the drainage pattern, to yield the direct runoff hydrograph of the Rock River near the project site. Although Sub-basin I has a relatively large area, its runoff contributes a relatively small amount to the flood peak because of its large lag time and because it is extensively regulated by lakes. A base flow of 3400 cfs was added to the direct runoff hydrograph to yield the total SPF hydrograph. The base flow was selected as the average of the lowest daily flow in the month of highest flow for each year during the periods 1947-49 and 1956-71. Daily stream flow data at Como was used and was transposed to the intake site.

The Muskingum method was used for channel routing. The routing parameter was approximated by travel time determined by the U.S. Army Corps of Engineers (Reference 5), and the parameter X was selected as 0.2. The sensitivity of the routing effect on the parameter X was examined by repeating the channel routings for various values of X. The difference between discharges at the site is only 1% for X equal to 0 and 0.2. Use of X = 0.2 yields the higher discharge.

Figure 2.4-15 shows the resulting standard project flood hydrograph for Rock River near the project site. The peak discharge of the standard project flood at the site is 154,000 cfs.

Dams upstream are low, generally 5 to 10 feet, and have little storage in their pools. Therefore, the failure of these dams during large flood would not measurably increase flood levels.

2.4.3.6 Probable Maximum Flood Flow

The peak discharge of the probable maximum flood for the Rock River at the intake was calculated to be 308,000 cfs. This discharge was derived by doubling the peak discharge of the standard project flood.

2.4.3.7 Combined Event Flood on the Rock River

The combined event flood is defined as a flood on the Rock River having 1 x 10^{-6} annual probability of exceedance at a 90% confidence level. The peak discharge of the combined event flood at the site as computed from flood data from 1915 to 1974 is 178,000 cfs and the flood level corresponding to this discharge is 698.68 feet. When the flood data for 1975 and 1976 are included in the analysis, the resulting combined event flood at the site is calculated as 176,900 cfs. The river screen house is the only structure that could be affected by flooding on the Rock River and is designed for the combined event flood.

2.4.3.7.1 Frequency Analysis

The annual maximum flood flows at the site from 1915 to 1974 were estimated by transposing the flows at Como by the square root of the drainage area ratio. The U.S. Water Resources Council (Reference 7) has recommended that Log-Pearson Type III distribution should be used as the uniform method for flood frequency analysis.

The procedure for calculating the flood frequencies has been described by Beard (Reference 8) and U.S. Water Resources Council (Reference 7). This procedure is adopted herein. The annual maximum flood discharges at the site, Q_{site}, were converted into logarithms of flows. For this set of data, the mean (M), standard deviation (S), and skew coefficient (g) were calculated using the following equations:

$$M = \sum X/N \tag{2.4-3}$$

$$S = \sqrt{\frac{\Sigma X^2 - (\Sigma X)^2 / N}{N - 1}} \tag{2.4-4}$$

$$g = \frac{N \, \Sigma \, (X - M)^3}{(N - 1)\,(N - 2) S^3} \tag{2.4-5}$$

in which X = logarithm of the magnitude of flood (10^3 cfs), and

N = Number of years of record.

The values of M, S, and g are shown in Table 2.4-11. The flood discharges Q_{site} (in logarithmic units) for difference mean recurrence intervals are obtained using the following equation:

$$\text{Log } Q_{site} = M + kS \tag{2.4-6}$$

The values of the constant k for any given skew coefficient are tabulated by Harter (Reference 9) for specified mean recurrence interval.

2.4.3.7.2 Selection of Parameters

The frequency analysis using Log-Pearson Type III distribution relies on the statistical parameters of annual maximum flood data at the site. The values of mean and standard deviation calculated from the data are used. The uncertainties in the discharge resulting from the use of sample mean (M) and sample standard deviation (S) are accounted for when confidence levels are evaluated.

In the frequency analysis, it is recognized that the calculated skew coefficient of the station record is sensitive to the length of record and the presence of extreme events in the record. with longer station record and the absence of extreme events in the record, the skew computed from individual station data is more reliable.

At Como gauging station, a record of 60 years of data is available; and the record does not contain any extreme events. Hence, the skew coefficient obtained from such data is reliable. Based on the 60 years of data at the Como gauging station, the station skew coefficient is -0.75.

Consideration is also given to regionalized skew coefficient in selecting an appropriate value for the skew. There is no published information concerning the regionalized skew coefficient for the Rock River Basin either with the Corps of Engineers or with the U.S. Geological Survey. However, Hardison (Reference 10) has published generalized skew coefficients of annual floods for the entire United States. For Illinois, he has considered 42 stations, each having a record of at least 25 annual flood peaks through 1967. The average values of the skew coefficient for the State of Illinois given by Hardison is -0.57. From the isopleths of the generalized skew coefficients, the value for Rock River at Como is determined as -0.50.

Contacts were made with Corps of Engineers and U.S. Geological Survey to seek their opinion concerning the appropriate skew coefficient to be used for this station. U.S. Geological Survey (Champaign, Illinois) suggested using the Como Station skew coefficient in view of the long record at the station. Corps of Engineers' (Chicago, Illinois and Washington, D.C.) opinion was to use the regionalized skew coefficient. If regional skew coefficient is not available, they recommend using the station skew coefficient provided the station has a long record.

In view of the foregoing and the record at Como that does not contain extreme events, it is justifiable to use the station skew coefficient of -0.75. However, the regional skew coefficient of -0.50 is used in this analysis as a more conservative basis.

2.4.3.7.3 Confidence Level

The discharge for any specified mean recurrence interval calculated using Equation 2.4-6 is subject to variations because the sample statistical parameters' mean and standard deviation, are generally different from the population mean (μ) and population standard deviation (σ), respectively. The predicted discharge Q_{site} varies about the actual Q_{site}, described by a noncentral t distribution. The 10% error limits in standard deviation units are calculated using the tables given by Resnikoff and Lieberman (Reference 11) and Johnson and Welch (Reference 12).

With the record of 60 years available at the site, these error limits were calculated employing the normal approximation suggested by the above authors.

Table 2.4-11 shows the flood discharges in the Rock River at Byron site for specified mean recurrence intervals for a selected confidence coefficient of 90%. Figure 2.4-16 shows these results plotted. The peak discharge of the combined event flood was estimated to be 178,000 cfs.

2.4.3.8 Water Level

The water surface elevations of the Rock River near the project site corresponding to the PMF, combined event flood, SPF, flood of record, mean annual flow, and lowest 1-day flow were determined by computer backwater analysis (Reference 13) and are tabulated in Table 2.4-12. Backwater profiles are shown in Figure 2.4-17. Input data to the program were river cross sections, distance between sections, starting elevations, roughness coefficients, and flow rate for each reach.

The overbank elevations of the cross sections were measured from USGS maps with a scale of 1:62500 and contour interval of 20 feet; channel data were measured from a set of U.S. Army Corps of Engineers maps surveyed during 1912-1913 showing river depths, with a scale of 1:4800. Distances between sections were measured from the same maps for channel and left and right overbank reaches.

Rock River and flood plain cross sections in the vicinity of the site are shown in Figure 2.4-18. The cross sections were developed primarily on the basis of 1912-1913 maps, prepared by the U.S. Army Corps of Engineers, which provide detailed channel depths not shown on the more recent U.S. Geological Survey maps published during the period 1917 through 1950. The USGS maps were used to extend the sections to higher elevations.

A comparison of the 1912-1913 maps, with air photos taken in April 1971, indicated that there were no major channel shifts between 1912 and 1971. This comparison was made for the 20-mile reach between a point 3 miles upstream of Byron, Illinois, and a point 3 miles downstream from Oregon, Illinois. A comparison of cross sections based on the 1912-1913 map, Figure 2.4-19, with cross sections recently surveyed near the intake, Figure 2.4-20 showed that the level of the river channel is essentially unchanged, as shown by the sections plotted on Figure 2.4-21. Based on the above comparisons, the 1912-1913 maps are considered to adequately reflect current river conditions.

The elevation-discharge relation for a section on a large river of flat slope may be determined by the channel conditions over a considerable downstream distance. Consequently, the first cross section was taken at Sterling just upstream from the upper dam, 41 miles below the site area. Cross sections were taken at 1 mile intervals upstream to approximately 10 miles below the intake point, at 1/2 mile intervals for the next 5 miles, and at 1/4 mile intervals for the last 5 miles to the site area.

Starting elevations upstream from the Sterling dam were determined by assuming normal depth of flow. The resulting calculated water levels at the site are not influenced by the selection of initial water surface elevation. Because of the reach and influence of the Dixon and Oregon dams, a difference in starting elevation of even as much as 10 feet during floods would decrease to about 0.1 foot at the site.

Flow roughness coefficients for channel and overbank flow were estimated on the basis of the coefficients determined by the U.S. Army Corps of Engineers for the Rock River. The coefficients and contraction and expansion coefficients are listed in Table 2.4-13. Except for flow rate and starting depth, the same data were common to all backwater computations. Based upon these common river characteristics and a sufficient number of flow rates and starting elevations, a rating curve at the intake was developed as shown in Figure 2.4-22.

2.4.3.9 Coincident Wind Wave Activity

The combined event flood (CEF) discharge for the Rock River at the intake point was calculated to be 178,000 cfs.

The flood level corresponding to this discharge is 698.68 feet above mean sea level. The significant and maximum wave effects of a coincident 40-mph overland wind were superimposed on the combined flood water level at the river screen house. Figure 2.4-23 shows the fetch at the screen house on the Rock River. The wave runups including setups were calculated based on a water condition and are 2.77 feet and 4.71 feet for the significant and maximum waves, respectively. Superimposing the runup values on the combined event flood level at the screen house resulting in a wave runup elevation of 701.45 feet for significant waves and elevation 703.39 feet for maximum waves. The pertinent wind wave parameters used in computing wind wave characteristics are tabulated in Table 2.4-14.

In order to prevent the river screen house from flooding due to the maximum wave runup, the safety-related equipment at the River Screen House is at an elevation of 702 feet and is enclosed by a 4-foot high fire wall.

The hydrostatic and hydrodynamic forces used in the analysis of the river screen house were determined using the parameters listed in Table 2.4-30. Forces were computed using methods in "Shore Protection Manual of the Corps of Engineers, 1977." The load combinations addressed were as follows:

Extreme Environmental Condition:

1. OBE and CEF

2. SSE and FOR (Flood of Record)

3. OBE and Breaking Wave

Severe Environmental Condition:

1. OBE and FOR

The stress level in the structural elements designed for these hydrostatic and hydrodynamic forces are within the design basis allowables.

2.4.4 Potential Dam Failures, Seismically Induced

The river screen house, which provides the makeup for the essential service cooling towers, is the only structure which could be affected by the failure of an upstream or downstream dam. Failure of the Rock River dams at Rockford (13 feet high), 22 miles upstream, and at Rockton (11 feet), 44 miles upstream, would create minor flood waves which would dissipate before reaching the site area. Failure of upstream or downstream dams during flood conditions would not affect the supply of makeup water to the plant.

2.4.5 Probable Maximum Surge and Seiche Flooding

Surge and seiche flooding is not a design consideration as there are no large bodies of water near the site.

2.4.6 Probable Maximum Tsunami Flooding

Tsunami flooding is not a design consideration as the site is not near a coastal area.

2.4.7 Ice Effects

Seven of the ten largest floods tabulated in Table 2.4-5 occurred in March and three in April. Ice cover and snowmelt are generally at their greatest during these months. This combination of conditions indicates the potential for ice jams at bridges, islands, channel bends, and other natural or man-made flow constrictions during late winter and early spring periods of high water. Newspaper articles, historical documents, and records on past floods of the Rock River describe the frequent occurrence of ice jams (Reference 2).

The flood of February 1937 reached a stage of 14.6 feet due to backwater from ice and is considered to be the worst ice jam flood on the Rock River at Rockton. More recently, in the early spring of 1971, there was major ice flooding approaching the magnitude of the 1937 ice floods; ice jams occurred all the way from Rockford downstream to Grand Detour, 15 miles below the site area. However, the 1937 flood stage was 0.94 feet lower than the second largest recorded flood of March 1975 at Rockton. The computed March 1975 flood reached an elevation of 679 feet at the intake. Since the safety-related equipment at the river screen house is at elevation 702.0 feet, ice-induced high flood levels will not have an adverse effect on the performance of the river screen house. Similarly, ice jams cannot create low water levels at the river screen house since the water level at the screen house is controlled by the downstream dam at Oregon.

2.4.8 Cooling Water Canals and Reservoirs

The only canal associated with the plant is an open flume which returns the circulating water from the natural draft cooling towers to the circulating water pumphouse. The flume is not safety-related and has a 2-foot freeboard. There are no reservoirs for the plant.

Makeup water is withdrawn from the Rock River and is pumped uphill to the plant in three pipelines. Two 12-inch essential service water makeup lines supply makeup to the essential service water cooling towers and one 48-inch circulating water makeup line supplies makeup to the intake bay of the circulating water pumphouse. Plant blowdown water from the circulating water system is discharged back into the Rock River in a 30-inch pipeline parallel to the makeup pipelines.

2.4.9 Channel Diversions

Due to the great width of the Rock River and the relatively flat surrounding terrain, there is little possibility that rock falls, ice jams, or subsidence could completely divert the flow away from the makeup water intake. The minimum daily flow of record (1915-1971) in the Rock River at the intake is estimated to be 400 ft^3/sec. Low flows are usually associated with the months of August, September, and October. No ice-induced low flow levels at the intake were reported. The intake is designed to prevent ice jamming against it and cutting off inflow. In order to prevent blockage of the intake structure by the accumulation of floating sheet ice, a floating boom and sheet piling are installed to deflect sheet ice away from the intake structure bar grills, allowing the river current to carry the sheet ice downstream. The upstream sheet piling provides a continuous shoreline to increase the river current past the intake, minimizing the potential for an ice jam at the bar grills. Even if the river flow were temporarily cut off, makeup for the essential service cooling towers would still be available from groundwater wells at the plant site.

In order to prevent the blockage of the intake structure by the aggradation of sediment and to maintain a permanent connection to the main deep channel of the river, a sediment management system is installed. The system, which was rigorously tested using a scale hydraulic model at the University of Iowa's Institute of Hydraulic Research (Reference 37), consists of submerged upstream wing dams and precast concrete vanes in front of the intake. These structures plus river configuration maintain a connection between the main deep channel of the river and the intake structure. The system also prevents sediment from accumulating at the face of the intake. It ensures that sediment induced channel diversions will not occur.

Minimum pump submergence requirement is 22-1/2 inches. The pump intake is about 15-1/2 inches above the bottom of the sump which is at elevation 660 feet 6 inches.

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2.4.10 Flood Protection Requirements

The river screen house, which provides the makeup for the essential service cooling towers, is the only structure which could be affected by floods on the Rock River. This structure is designed for the combined event flood and waves produced by a 40 mph wind. The engine for the essential service water makeup pumps is mounted on its sub-base at elevation 703 feet 8-1/2 inches. The engine shaft centerline is at elevation 705 feet 4 inches and the lower battery post elevation is approximately 703 feet 8 inches. It is anticipated that the latter elevation would be limiting under flood conditions. This is above the combined event plus maximum wave run up elevation and is anticipated to be the elevation at which the engine would stop.

The engine-driven essential service water makeup pumps will automatically start and continue to operate regardless of whether offsite power is available or not.

In the unlikely event that the engines are rendered inoperable by a flood level in excess of 703 feet 8 inches, the onsite water wells will be powered from their respective Unit 1 ESF buses. These wells will then provide the makeup for the essential service cooling towers.

The probable maximum flood from the Rock River cannot affect the other safety-related structures.

2.4.11 Low Water Considerations

2.4.11.1 Low Flow in the Rock River

Low flow frequency analyses for the Rock River at Rockton and at Como were made using the Log-Pearson Type III distribution (Reference 14). Flows at the intake were interpolated using Equation 2.4-1 in Subsection 2.4.2.

Table 2.4-15 gives flows in the Rock River at the intake for various combinations of duration and recurrence interval. Considerations of downstream dam failures are included in Subsection 2.4.11.5.

2.4.11.2 Low Water Resulting from Surges, Seiches, or Tsunami

Low water conditions resulting from surges, seiches, or tsunami are not design considerations because there are no large bodies of water near the site, nor is the site near a coastal area.

2.4.11.3 Historical Low Water

A minimum daily flow of 440 cfs was recorded at Como on August 20, 1934. The historical 1-day low flow at the intake is estimated to be 400 cfs and has a recurrence interval of more than 100 years. The corresponding river elevation at the intake is 670.4 feet.

2.4.11.4 Future Controls

Future upstream uses of Rock River water are not expected to lower minimum flows. Since most communities derive their water supply from groundwater, the trend will be toward higher future minimum flows due to increased sewage effluent discharges.

2.4.11.5 Plant Requirements

The circulating water makeup is withdrawn from the Rock River. The maximum water requirement for plant use is 107 cfs. Actual use might be less depending on plant operating loads and seasonal variability of evaporation and blowdown losses. Since only 61 cfs are used up due to evaporation and drift, 46 cfs are returned to the Rock River. Thus, the net withdrawal rate is 61 cfs. These requirements include makeup water for the essential service cooling towers, of which 2 cfs are for evaporation and drift losses and 1.33 cfs are for blowdown.

Backup essential service water makeup may be withdrawn from the Rock River. The maximum quantity of water withdrawn is 3580 gpm (two pumps operating at approximately 1790 gpm for each pump).

The required SX makeup rate varies depending on the evaporation rate, blowdown rate, drift, and possible use of essential service water as the backup supply for auxiliary feedwater. SX inventory losses during an accident may initially exceed the available makeup because of the high evaporation load due to the heat rejected to the UHS and/or possible AF flow. The mismatch between the makeup rate and inventory losses becomes smaller as blowdown is isolated, the heat load drops, and AF flow drops with cooldown of the plant. Adequate water is available in the SX cooling tower basins to accommodate the draw down of inventory until the makeup rate exceeds the inventory losses and the basins begin to refill.

In the unlikely event that emergency cooling water requirements cannot be satisfied by makeup from the Rock River, deep wells will provide makeup to the essential service water cooling tower.

A summary of the cooling water capabilities of various pumps and wells is provided in Table 2.4-16.

Table 2.4-17 illustrates the required minimum safety-related cooling water flow, the sump invert elevation and configuration, the minimum design operating level, and the required minimum pump submergence.

The essential service water makeup pumps are capable of supplying sufficient water during periods of low water resulting from the 1-day 100-year drought. From Table 2.4-15, the 1-day 100-year low flow at the intake is 454 cfs. The corresponding water surface elevation is 670.4 feet.

Backwater analyses for low-flow conditions in the Rock River indicate that a reduction of 10% in the river discharge would result in only negligible changes of water-surface levels at the pumping site and downstream. Backwater profiles were computed (Reference 13) for discharge conditions shown in Table 2.4-18 for the river reach from Sterling to the pumping site, a distance of 41 miles. Above the dam at Oregon, changes in water-surface levels due to withdrawal of 10% of the low-flow discharge would be 0.03 foot or less. Between the dams at Sterling and Oregon, the average differences in water levels would range from 0.05 to 0.09 foot, as shown in Table 2.4-18.

Water levels at Como, with and without cooling water withdrawals, were estimated from the USGS rating table for the Como gauge 3 miles downstream from the dam at Sterling. With 10% withdrawal, the change in stage would be approximately 0.08 foot at Como for the low-flow conditions listed in the table. This change confirmed water levels derived by backwater analyses since the water surface elevation at Como is not controlled by a small dam as it is above Sterling, Dixon, and Oregon.

An extremely low water level could possibly occur through combination of low river discharge and breaching of the Oregon dam 5 miles downstream. Since the lowest point on the river bottom at the intake is about 10 feet below the dam's crest, removal of the impounding effect of the dam during low flow would lower the water surface at the intake. Consequently, studies were made to determine that level. The same computer model was used as described in Subsection 2.4.3 with a channel "n" value of 0.032 and a river flow of 400 cfs, the 1-day lowest flow at the site area. The resulting water-surface elevation at the intake would be 664 feet 4 inches. As indicated in Table 2.4-17, essential service water makeup pumps can supply makeup water even under this extreme condition.

Blowdown from the plant cooling towers is discharged into the river in the discharge flume. The thermal mixing zone will be consistent with regulations specified by the Illinois Pollution Control Board. Design bases for effluent submergence, mixing, and dispersion are given in Subsection 2.4.12.

The station service water systems and the essential service water system are described in Subsection 9.2.1.

2.4.11.6 Heat Sink Dependability Requirements

The design of the ultimate heat sink for the Byron Station follows Regulatory Position 2C of NRC Regulatory Guide 1.27 (Revision 2, January 1976) in that all reasonably probable combinations of less severe natural phenomena are accounted for. As explained in this section, the simultaneous occurrence of a flood greater than the combined event flood and of an earthquake is not a concern since an alternate source of makeup water is available to the heat sink from seismically qualified deep wells.

The ultimate heat sink for the station consists of two redundant essential service cooling towers and basins and their associated makeup systems. The design of the towers, basins, and makeup lines are Safety Category I, Quality Group C and shall withstand a safe shutdown earthquake. There are two sources of makeup water for the tower basins: the Rock River and groundwater from wells. During the normal operation of an essential service cooling tower, makeup water from the Rock River reaches the tower makeup line via the circulating water system. Makeup is also available from the Rock River through one or both of the two essential service water makeup pumps located in the river screen house. The screen house and pumps are Safety Category I design and have been located to withstand the Rock River combined event flood with waves generated by a 40 mph wind. Each makeup pump is powered by a separate diesel drive and discharges to an essential service cooling tower makeup line. The two essential service cooling tower basins are connected via a Seismic Category I overflow trough.

Makeup can be supplied to the towers by one or more onsite deep wells of Safety Category II, Quality Group D design. These wells have been qualified for the safe shutdown earthquake and are protected from tornado wind pressure and missiles. The well heads, located approximately 200 feet elevation above the river and approximately 3 miles from the river are enclosed in heated buildings to provide freeze protection.

The onsite wells at Byron are cased with ASTM A53 Grade B steel casing (3/8 inch thick) through the soil and dolomite strata into sandstone. Individual lengths of well casing were welded together when installed. The annular spaces between the bore holes and the well casings were grouted with concrete grout from the bottom upward in order to seat the casings into the bedrock and to provide seals preventing the movement of soil or surface contaminants into the wells. The production portion of the wells consists of uncased, open boreholes which were over-pumped after completion to remove any loose rock or drill cuttings. The type of well construction, with the length of casing welded together and seated into the bedrock, provides the maximum strength for any groundwater well. Municipal or large-volume industrial wells in northern Illinois are generally of similar or lower quality construction.

During pump testing of these wells, some caving of sandstone was observed which might interfere with the pump performance and reduce the productivity of the well. The actual zone of caving was determined by caliper-logging of the borehole and the wells were deepened to allow for any debris to collect at the bottom and still assure adequate yield. A smaller diameter casing was extended deeper into the well placing the pump setting within the cased portion of the well. This prevents any caved material from damaging the pump. With these modifications, the wells assure adequate supply to the UHS when needed.

The design elevation of the pump invert which supplies makeup to the essential service cooling tower basins from the Rock River has been based on the postulated low water elevation resulting from the breaching of the Oregon Dam during the historic low flow period. This occurrence would result with a river flow of 400 cfs, a water elevation of 664 feet 4 inches. The historic low flow of the Rock River recorded in 1939 at Como, Illinois was 440 cfs. In addition, under these conditions, an alternate source of makeup water is available from the seismically qualified deep wells.

Analyses were performed to demonstrate that makeup water is available for 30 days and beyond at a rate which satisfies the most severe design basis as set forth in NRC Regulatory Guide 1.27 positions C.1.a and C.1.b. The analyses were based on the scenarios and assumptions described in Section 9.2.5.3.5.

The connections between the essential service water cooling towers and the auxiliary feedwater train are provided with normally closed motor-operated valves. Protection against single active or passive failures is provided by the redundancy of the essential service water system.

An analysis of the impact of supplying water to the auxiliary feedwater train from the ultimate heat sink indicates that adequate basin inventory and makeup exists to supply water to the AF system, concurrent with shutdown from a two unit plant trip from full power, or a LOOP/LOCA on one unit and safe shutdown of the second unit.

2.4.11.6.1 Earthquake Analysis of Deep Wells

The deep wells and related components have been qualified for the safe shutdown earthquake (SSE) based on information contained in Reference 36. The qualification consisted of two parts: 1) a literature review of several worldwide earthquakes and the performance of wells and 2) a seismic analysis of the wells and components. From the literature review, six worldwide earthquakes, with strong ground motion which was 10 to 150 times stronger than the Byron SSE, were selected and the performance of wells exposed to this motion was compared to the Byron deep wells. The worldwide experience indicates that deep, well-constructed, cased wells, either in alluvial soils or rock, have withstood a wide range of earthquake ground motions provided that the wells are not subjected to fault displacements, ground separation, landslide shear, or lateral spreading of liquefied soils. The Byron deep wells are cased wells grouted in rock. Based upon the extensive geologic and seismological investigation of the site, no active faulting has been found within 200 miles, therefore, fault shears and ground separations will not occur. Thus, the Byron deep wells will not experience damage or impairment of production due to the SSE.

The seismic analyses of the components of the deep well system were performed based on technically conservative and acceptable procedures generally used for similar seismic safety-related components in the nuclear power industry. The items analyzed are: the uncased borehole cavity, the well casing, the discharge pipe within the casing, the motor and the pump, the buried discharge pipe, the buried concrete ductrun, and the pump enclosure structure. The results of the analyses show that strains and stresses induced in these components, during the safe shutdown earthquake (including the other normal loads), are well within the corresponding allowables.

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2.4.11.6.2 Combined Event Considerations

The 30-day 100-year recurrence drought flow at the intake is 739 cfs (Table 2.4-15). The corresponding water surface elevation at the intake with the Oregon Dam in place is 670.6 feet. The invert of the intake is at elevation 663.5 feet; thus, a water depth of 7.1 feet is available.

A 500-year seismic event at the site corresponds to a maximum horizontal ground acceleration of 0.05g. The assumption of a failure of the Oregon Dam subject to this level of seismic loading is extremely conservative. However, if such a failure is postulated coincident with a 30-day 100-year recurrence drought, the water surface elevation at the intake would be 665 feet, providing a depth of 1.5 feet of water on the floor of the intake.

A simplified evaluation of the seismic resistance of the Oregon Dam was made using data from Reference 30. The lateral resistance of sheet piling, liquefaction potential of the subsurface sand (Reference 31), and the stability of the dam were evaluated. On a conservative basis, it was determined that the dam can sustain a maximum horizontal ground acceleration of at least 0.1g without failure.

From data presented in a recent study by Dames and Moore (Reference 32), it is estimated that the probability of an earthquake having a maximum acceleration equal to or greater than 0.1g is about 0.3×10^{-3} per year. Hence, the probability of occurrence of an earthquake causing failure of the Oregon Dam is much less than 1/500.

It is estimated that the temperature would be low enough for ice formation and accumulation, at most, for 2 months of the year. Therefore, the probability of not having the Rock River to provide makeup water during a 30-day 100-year drought coincident with an earthquake having a maximum acceleration of 0.1g would be no greater than:

$$P = (0.3 \times 10^{-3})\ (1/100)\ (2/12)$$

$$= 5.0 \times 10^{-7} \text{ occurrences/year.}$$

The sediment management system installed in the Rock River as described in Section 2.4.9 is designed to maintain the bed levels at the intake structure below the intake sill elevation an to maintain a channel connection to the main deep channel of the river. It is also designed to maintain the channel connection below the intake sill elevation along its entire length. As such, even during a combined seismic dam failure and 30-day, 100-year drought (with a water level as low as 665'-0"), sufficient water will be available at the intake structure.

The Iowa Institute of Hydraulic Research measured sediment concentration at the intake (Reference 33). The bed load at the intake mainly consists of fine sands. The particle size distribution is fairly uniform with a d_{50} = 0.4 mm. The suspended sediments are entirely in the fine silt to clay particles size range. Table 2.4-31 provides suspended sediment concentrations at the intake.

The U.S. Geological Survey (USGS) has published suspended sediment data for Rock River at Joslin for the water years 1975-1979. The drainage area of the Rock River at Joslin is 9549 square miles. Ninety percent of the suspended sediment is finer than 0.062 mm.

About 90% of the sediment carried by the river constitutes suspended sediment and it is kept in suspension due to the turbulence of the river. The sediment management system ensures that the suspended sediment and bed load will not block the intake structure by creating a local scour (Reference 37). This scouring action carries suspended sediment and bed load away from the channel connection and intake structure. Therefore, the blockage of the intake with suspended sediment and bed load is not probable. Since the Rock River is considered to be stable, blockage of the intake with bed load due to large scale river meandering is not probable.

There is no data available regarding ice thickness on the Rock River. USGS indicated that maximum thickness of ice observed at the discharge measuring stations at Rockton, Byron, and Como was 1.9 feet during the 1978-79 winter which is one of the severest winters of record.

The thickness of ice on lakes can be predicted by using the following equation (Reference 34):

$$h_i = L\,(1.06\ \ \sqrt{S}) \qquad (2.4\text{-}10)$$

where:

h_i = the ice thickness in inches,

L = the coefficient of snow cover and location conditions, and

S = the accumulated degree-days since freezeup, based on °F below freezing.

The coefficient L is 0.75 to 0.65 for medium size lakes with moderate snow cover. Average annual snowfall at the Byron site is about 28 inches. Hence, L is taken as 0.65. The average annual freezing degree-days at Rockford, Illinois are 1123°F-day. The winter year 1976/1977 was the coldest year on record in northern Illinois. The corresponding freezing degree-days at Rockford are 1727°F-day. Equation 2.4-10 gives the thickness of ice cover for a lake at 23 inches and 29 inches, for an average year and for the coldest year (1976-1977), respectively. However, these values are for a lake and not directly applicable to rivers. For rivers, the flow resistance reduces the thickness of the ice. Freezeup starts in late November and reaches a maximum in March. Based on historic flow data, minimum flow occurs during August-September. During winter, the flow gradually increases from November to March. The minimum monthly flows and the average monthly flows at the intake based on the recorded flow data at Como gauging station are given in Table 2.4-32.

From the above discussion, it is clear that ice (maximum thickness is 29 inches) does not block the intake since the depth of water available is 7.1 feet under 30-day 100-year low flow conditions.

Frazzle ice is a term referring to small ice particles which may form at the water surface if the air temperature is quite low and the mixing and conductivity of the water is insufficient to prevent a slight supercooling of the water surface. Based on operating experience, frazzle ice is not expected to affect the operation of the river intake at the Byron Station. To help prevent frazzle ice formation on the intake bar grills and traveling screens, circulating water system blowdown can be directed from the blowdown outfall structure to the intake of the river screen house. The line is routed underwater and outside of the bar grills. Nozzles direct the water up and along the bar grills. The warm blowdown water mixes with the incoming Rock River water to prevent formation of frazzle ice on the bar grills and traveling screens during winter operation.

Ice and sediment cannot block the intake because of the availability of a 7.1-foot depth of water. In the event of Oregon dam failure during a low flow period, the seismically qualified deep wells will supply the essential service water to the plant.

2.4.11.6.3 Heat Sink Dependability

Emergency cooling sources and associated principal facilities comprise the Rock River, the river screen house, the essential service water cooling towers, the groundwater wells, and attendant distribution systems. The river screen house is a Seismic Category I structure protected from flooding up to the combined event flood. Groundwater wells, located at the plant site, are above the probable maximum flood water levels. The groundwater wells will be used for makeup to the essential service water cooling towers whenever the river screen house is unavailable as a result of flooding conditions which exceed the combined event flood.

The ultimate heat sink consists of two essential service water mechanical draft cooling towers and the makeup system to these cooling towers. Applicable design considerations and descriptions of the ultimate heat sink are presented in Subsection 9.2.5.

In order to prevent the blockage of the intake structure by the aggradation of sediment and to maintain a permanent connection to the main deep channel of the river, a sediment management system is installed. The system, which was rigorously tested using a scale hydraulic model at the University of Iowa's Institute of Hydraulic Research (Reference 37), consists of submerged upstream wing dams and precast concrete vanes in front of the intake.

These structures plus river configuration maintain a connection between the main deep channel of the river and the intake structure. The system prevents sediment from accumulating at the face of the intake.

design during low river levels. The river conditions which could produce a similar occurrence do not exist at the Byron intake structures.

Table 2.4-19 shows the monthly average mean and minimum flows of the Rock River at the intake for the period starting from 1967 to 1976. The flows are transposed to the intake from those at Rockton by the drainage area ratio. Low flows occur during the months of August, September, and October. During March, April, and May, when ice jams could occur, high flows persists. Hence, it is not likely that ice effects will induce low flow levels at the intake. The ultimate heat sink will be able to withdraw water from the Rock River during heavy winter icing period.

2.4.12 Dispersion, Dilution, and Travel Times of Accidental Releases of Liquid Effluents in Surface Water

The boron recycle holdup tank is the largest tank containing radioactive effluents. The inventory of radionuclides contained in this tank is given in Table 2.4-20. This tank is located in a portion of the Seismic Category I auxiliary building where the floor elevation is 815.0 feet. The plant grade elevation is 869.0 feet. As stated in Subsection 2.4.13.3, the groundwater elevation at the plant site is assumed to be 799.0 feet. Therefore, any postulated accidental release of effluents through postulated cracks in this building would leak into the surrounding soil media and travel through the underlying aquifer prior to discharge in a surface water body.

There are only four outside surface tanks which may contain radioactive liquids. These are the two identical 450,000-gallon capacity refueling water storage tanks and the two identical 500,000-gallon capacity primary water storage tanks.

The refueling water storage tanks are located near the fuel handling building on a 6-foot thick reinforced concrete mat. The tanks are reinforced cylindrical structures consisting of 2-foot thick walls lined on the inside with a 1/4-inch stainless steel liner. The refueling water storage tanks are considered as Category I structures (and leaktight) and are discussed in Subsection 3.8.4.

The primary water storage tanks are located near the turbine building. The tanks and tank foundation are designed for the seismic load condition for OBE and SSE.

The nearest point where the effluents moving with the ambient groundwater appear as surface flow is a spring located 3630 feet northwest of the auxiliary building. Water flowing from this spring joins the Woodland Creek through a tributary. The Woodland Creek, in turn, joins the Rock River.

The hydraulic gradient between the plant site and the nearest spring is 0.011. Other characteristics of the surrounding soil and rock mass are described in Subsection 2.4.13.3.

Conservatively assuming that only 10% of the saturated thickness of the aquifer between the plant building and the spring would contribute to the dilution of the effluents with the ambient groundwater, the minimum available dilution factor would be 655. The travel time of the effluents from the auxiliary building to the spring is estimated to be 64.12 years. The available dilution factor and the large travel time would reduce the concentrations of all the radionuclides listed in Table 2.4-20 to well below the 10 CFR 20 limits before their arrival at the spring. Additional dilution would be available in the Rock River where the 7-day 10-year low flow is 925 cfs and the nearest user is located about 22 miles downstream.

2.4.13 Groundwater

Character, water quality, yield, and depth of hydrogeologic units are discussed with emphasis on the major aquifers. Groundwater use and supply are also discussed.

The site work, particularly the installation and testing of two groundwater wells, has confirmed that the site hydrogeologic conditions were as anticipated from the PSAR-stage investigations.

2.4.13.1 Description and Onsite Use

2.4.13.1.1 Regional Conditions

The site area is located on an upland south of the Rock River. The area is underlain by the Ordovician-age Galena-Platteville dolomites and the older Ordovician-age Glenwood Formation and St. Peter Sandstone (Reference 16). The bedrock dips gently to the southeast at 15 to 25 feet per mile, although local warping is common. The bedrock is covered in most places by a thin mantle of glacial drift. A generalized stratigraphic column showing regional hydrogeologic units is presented in Figure 2.4-24. The most important aquifer in the region is the Cambrian-Ordovician Aquifer, made up of all bedrock between the top of the Galena-Platteville dolomites and the top of the Eau Claire Formation. These strata are, in descending order, the Ordovician-age Galena, Platteville, Ancell, and Prairie du Chien Groups and the Eminence Formation, Potosi Dolomite, Franconia Formation, and Ironton and Galesville Sandstones. At the Byron Station, the Galena-Platteville Dolomites are separated from the missing Cambrian-Ordovician Aquifer by the Harmony Hill Shale Member of the Glenwood Formation. The Cambrian-Ordovician Aquifer is separated from the deeper Mt. Simon Aquifer by the shale beds of the Eau Claire Formation. Available data indicate that, on a regional basis, the entire sequence of strata above the Eau Claire Formation behaves hydraulically as one aquifer. In places, however, pressure heads between the waterbearing units differ, and the hydraulic connection is imperfect. The Maquoketa Shale Group is absent in the site area. The St. Peter, Ironton, and Galesville Sandstones are recharged from overlying glacial deposits in the central and western parts of northern Illinois where the Maquoketa Shale Group has been removed by erosion. These units are also recharged by vertical leakage through the Maquoketa Shale Group in northeastern Illinois and by through-flow from the outcrop area in southern Wisconsin.

2.4.13.1.2 Site Conditions

The four most significant hydrogeologic units at the site are the glacial drift, the Galena-Platteville dolomites, the sandstone units of the Cambrian-Ordovician Aquifer (the St. Peter, Ironton and Galesville Sandstones), and the Mt. Simon Sandstone. These units are discussed in the material which follows and are described in Table 2.4-21, a generalized hydrogeologic column at the site.

2.4.13.1.2.1 Glacial Drift

The site area is covered with a mantle of glacial drift (Reference 17) consisting mainly of glacial till covered by a few feet of loess (windblown silt). A study of borehole logs at the site indicates that the thickness of the drift at the site averages about 16 feet. Groundwater may occur in the drift perched upon the underlying bedrock. Limestone rock fragments in the drift impart hardness and alkalinity to water in shallow-dug or augered wells. The generally low permeability and thinness of the till precludes development of the drift by drilled wells. The drift is recharged by precipitation.

2.4.13.1.2.2 Galena and Platteville Groups

Beneath the thin mantle of drift, the area is underlain by dolomites and limestones of the Ordovician-age Galena and Platteville Groups. The dolomites are extensively fractured near the top, with solutionally enlarged openings in places but become dense at depth. In areas of thin drift, particularly where the till may be absent or sandy, water-table conditions may prevail. Study of borehole logs indicates that the Galena-Platteville dolomites range from about 100 to 225 feet thick at the site. The depth to the top of the dolomite equals the thickness of the drift.

A piezometric surface map, constructed from measured groundwater levels in onsite observation wells prior to plant construction, is presented in Figure 2.4-25. The piezometric surface map shows that Byron Station lies on a potentiometric high, with groundwater movement radially outward. The piezometric surface generally reflects the ground surface, as expected in a water-table aquifer.

In the site area the Galena-Platteville dolomites are recharged by precipitation through the overlying glacial drift and discharge into the Rock River and its associated tributaries and in shallow domestic wells.

Water from the Galena-Platteville dolomites in the area is generally hard. Therefore, although it might be possible to obtain 10 to 50 gpm of hard water from the limestone with less than 200 feet of drilling, deeper drilling for softer water in the sandstone might be more economical. Relatively low yields, water hardness, and susceptibility of the aquifer to contamination because of thin drift, fractures, and solution channels do not favor development of the Galena-Platteville dolomites.

2.4.13.1.2.3 St. Peter Sandstone

Below the Galena-Platteville dolomites are the thin shales, sandstones, and limestones of the Glenwood Formation. This unit grades down into the thick sandstones of the St. Peter Sandstone. The Ordovician-age St. Peter Sandstone is permeable and has a relatively uniform lithology throughout the area. In the regional area, the St. Peter Sandstone is discharged primarily through wells for small municipalities, subdivisions, parks, and several industries that have water requirements generally less than 200 gpm (Table 2.4-22).

Numerous wells in the site area obtain water from the St. Peter Sandstone. A well in the southeastern quarter of Section 12, just north of the site, encountered the Glenwood Formation at 225 feet and the St. Peter Sandstone at 235 feet. The well was finished in 65 feet of the sandstone. A well in the western part of the site encountered the Glenwood Formation at 183 feet, penetrated the St. Peter Sandstone at 205 feet, and was finished in sandstone at 275 feet.

The uneroded thickness of the Glenwood Formation in the area ranges from about 18 to 32 feet. The full thickness of the St. Peter Sandstone in the area is about 420 to 450 feet. Artesian conditions prevail, and the static level is above the top of the sandstone unit. Based on available data, the average specific capacity of 17 wells within 2 miles of the site finished in the St. Peter Sandstone is about 2.7 gpm/ft of drawdown.

If the overlying Galena-Platteville dolomites were cased off and the casing extended through the Glenwood Formation into the sandstone, softer water than in any of the overlying units should be obtained.

2.4.13.1.2.4 Ironton and Galesville Sandstones

Deep-well logs in the area suggest that the Prairie du Chien Group is locally missing and that Cambrian-age dolomites of the Eminence Formation, Potosi Dolomite, and Franconia Formation underlie the St. Peter Sandstone. These units have a combined thickness of about 225 feet in the regional area. The onsite deep wells confirm that the Prairie du Chien Group and Eminence Formation are missing at the site. The combined thickness of the Potosi Dolomite and Franconia Formation at the site is about 125 to 140 feet.

Below the Franconia Formation are the Ironton and Galesville Sandstones comprising a portion of the aquifer which is about 150 feet thick in the regional area. In the site area, the Ironton and Galesville Sandstones are about 105 to 115 feet thick. The sandstones are discharged primarily through wells to industries and municipalities. Regionally, the Ironton and Galesville Sandstones are considered the best bedrock aquifer in northern Illinois because of their consistent permeability and thickness (Reference 18 and Table 2.4-21). Yields on the order of hundreds of gallons per minute may be obtained from the Ironton and Galesville Sandstones in wells less than 1000 feet deep. As reflected by the relatively high specific capacities of the Byron Station wells, the Ironton and Galesville sandstones are a major water-producing zone in the Byron Station wells. Results of water quality analyses on samples collected from the onsite water supply wells are presented in Table 2.4-23.

2.4.13.1.2.5 Mt. Simon Sandstone

Below the Ironton and Galesville Sandstones is the Eau Claire Formation, about 405 feet thick. The basal part of the Eau Claire Formation and the underlying Mt. Simon Sandstone (which is about 1430 feet thick) form the basal Cambrianage Mt. Simon Aquifer. Wells yielding many hundreds of gallons per minute have been finished in the Mt. Simon Sandstone, which contains fresh water to depths of about 2000 feet (Reference 19). It is recharged in outcrop areas in Wisconsin and from vertical leakage from overlying aquifers. The Mt. Simon Aquifer is a major water-producing zone in the Byron Station wells. Water quality analyses for the wells are presented in Table 2.4-23.

2.4.13.1.3 Onsite Use

Two groundwater wells have been installed, developed, and tested at the plant for potable water supply and for demineralizer water. The groundwater will be filtered and stored in a 150,000-gallon storage tank prior to usage. The two plant wells will also serve as a backup system for makeup to the essential service water cooling towers.

Water for the demineralizer will be required at the rate of about 825 gpm for the first several months, and thereafter at an average rate of 450 gpm. On the basis of 50 gallons per capita per day and a design population for plant personnel of 300 people, the average annual groundwater potable supply required will be 15,000 gallons per day or about 10 gpm. Considering monthly, daily, and hourly above-average demand, the potable water supply developed will be on the order of 20 gpm. In the unlikely event that makeup to the essential service cooling tower is not available from the Rock River, the groundwater wells should provide a maximum of 1600 gpm for the duration of safe shutdown. The demineralizer and potable water use would not be additive to the 1600 gpm use as they would not be required simultaneously.

The two Byron Station deep wells (W-1 and W-2) were completed in the Ironton and Galesville Sandstones of the Cambrian-Ordovician Aquifer in 1974. The locations of these wells are shown in Figure 2.4-25. The wells were cased through the Galena-Platteville dolomites as indicated in Table 2.4-29. The water wells were open from the Ancell Group through the Ironton and Galesville Sandstones; as noted in Figure 2.4-24, the Ironton and Galesville Sandstone were the major producing zones. Grouting at the Byron Station did not extend into the formations open to the site water wells; therefore, the specific capacities are unaffected by onsite grouting.

The specific capacities obtained during well development pumping tests were 10.3 gpm/ft of drawdown at 620 gpm for 12 hours in Well 1 (east well) and 9.6 gpm/ft of drawdown at 1150 gpm for 24 hours in Well 2 (west well). The pumping rate for Well 1 was relatively constant for the initial 12 hours, then was varied between 433 and 930 gpm during the last 12 hours of the test. The pumping rate for Well 2 was relatively constant for the entire 24 hours.

In addition to the Byron Station water wells, a temporary water well (TW-1) was installed for construction supply. The location of this well is shown on Figure 2.4-25. The well construction was as follows: 16-inch diameter cased borehole to 30 feet, 15-inch diameter open borehole to 271 feet, and 12-inch diameter open borehole to 600 feet. This well was primarily open to the Galena-Platteville dolomites and the St. Peter Sandstone. The specific capacity of this well was 4.4 gpm/ft of drawdown at 840 gpm after 24 hours. The lower specific capacity may be attributed to the smaller borehole diameter and the lower productivity of the upper portions of the Cambrian-Ordovician Aquifer.

Two water wells were installed to provide water for the grouting operation. The wells are presently capped and are not in use. Well TW-2 is an 8-inch diameter well cased to 240 feet and open borehole to 505 feet. Well TW-3 is an 8-inch diameter well cased to 241.5 feet and open borehole to 500 feet. Pump settings are at 240 feet and 241.5 feet, respectively. Both wells produce water from the St. Peter Sandstone.

An aquifer pumping test was performed in November 1978 in order to demonstrate the ability of the station deep water wells to provide make-up to the essential service water system during the 30-day period for safe shutdown. The aquifer pumping test consisted of pumping water well W-1 at a continuous rate of 840 gpm while monitoring groundwater levels in water well W-2 (Ironton-Galesville Sandstones), the grouting supply well TW-2 (St. Peter Sandstone), and an observation well TW-4 installed in the Ironton-Galesville Sandstone approximately 300 feet from water well W-1 on the line connecting wells W-1, TW-2, and W-2. The aquifer pumping test consisted of a 22-hour pumping period followed by a recovery period of 1-1/2 hours. Locations of the wells are shown in Figure 2.4-25.

The aquifer pumping test results indicated that the aquifer transmissivity was 20,000 gpd/ft, the storage coefficient was 2.0×10^{-4}, and the specific capacity was 8.5 gpm/ft. The results also indicated that the pump setting would have to be deepened and that the specific capacity had decreased 15% since the well was first installed. On the basis of subsequent caliper logging and well depth measurements, the apparent decline in well productivity was attributed to caving from the St. Peter Sandstone with blockage of the productive aquifer in the lower portion of the well.

Well modifications performed in wells W-1 and W-2 after the 1978 pumping test consisted of reaming and casing-off the caving St. Peter Sandstone, deepening the wells through the Ironton-Galesville Sandstones and into the upper portion of the Mt. Simon Sandstone, increasing the pumps' lift-capacity, and lowering the pump settings by 100 feet. Deep well construction details are summarized in Table 2.4-29 and presented schematically in Figure 2.4-30. The modified water wells are open from the Franconia Formation, through the Ironton and Galesville Sandstones, and into the Mt. Simon Sandstone. As noted in Figure 2.4-25, the Ironton and Galesville Sandstones and the Mt. Simon Aquifer are the major producing zones.

The specific capacities obtained during well development pumping tests in the modified wells were 12.3 gpm/ft drawdown at 1330 gpm for 12 hours in Well 1 and 12.2 gpm/ft of drawdown at 1210 gpm for 9 hours in Well 2. Whereas the pumping rate for Well 1 was relatively constant for the entire 12 hour test, the pumping rate for well 2 was relatively constant for only the initial 9 hours, then was varied between 1200 and 1600 gpm during the last 3 hours of the test. The available drawdown in the two wells is approximately 125 feet based on a static water level of 225 feet and a pumping level of 350 feet.

An aquifer pumping test was performed in July 1980 after the well modifications were completed in order to demonstrate the ability of the modified station water wells to provide the required make-up to the essential service water system. The test consisted of pumping Well W-1 at 790 gpm for 24 hours. Drawdown and recovery were again measured in Well W-2. The test results indicated that the aquifer transmissivity is 40,000 gpd/ft, the storage coefficient is 2.5×10^{-4}, and the specific capacity is 13.2 gpm/ft.

There are no dewatering systems that will be used to permanently lower groundwater levels under safety or non-safety buildings.

2.4.13.2 Sources

2.4.13.2.1 Present Regional Groundwater Use

Most of the water for domestic, municipal, and industrial use in the region is obtained from groundwater sources. The major unit is the St. Peter Sandstone within the Cambrian-Ordovician Aquifer, although minor supplies commonly are obtained from the shallower glacial drift and dolomite aquifers.

There are seven public water supply systems within 10 miles of the plant site. All use groundwater wells for water supply. All obtain their supplies from the Cambrian-Ordovician Aquifer or the Mt. Simon Aquifer, which are dependable and capable of high yields. Table 2.4-22 lists and gives details of each of the seven public water supply systems (Reference 20). The locations of the public pumping centers are shown on Figure 2.4-26.

The total groundwater pumpage in Ogle County was 11.82 mgd in 1965, the highest during the period from 1960-1970. After 1965, pumpage declined at an average rate of 240,000 gpd per year until 1970. In 1970, total pumpage in the county was 10.62 mgd; of this total, 12% was from glacial drift wells, 20% was from shallow dolomite wells (Galena-Platteville dolomites), and 68% was from sandstones wells (Cambrian-Ordovician Aquifer). Pumpage for public supplies in 1970 accounted for 53% of the total groundwater withdrawal in Ogle County. All recorded public pumpage is from sandstone wells (Reference 21).

In the whole of Ogle County in 1970, 14% of the total pumpage was for industrial supply; of this pumpage, 93% was from sandstone wells. Sixteen industries pumped groundwater in 1970 (Reference 21).

Pumpage for domestic supplies in Ogle County decreased 11% from 3.95 mgd in 1960 to 3.53 mgd in 1970. Domestic pumpage was 33% of the total groundwater use in Ogle County (Reference 21).

The following summarizes the uses of the aquifers in Ogle County:

a. Glacial drift wells - Pumpage from glacial drift wells in Ogle County has been largely limited to domestic use.

b. Shallow dolomite wells - Total pumpage from shallow wells has been almost entirely for domestic supplies, with only limited pumpage for industrial supplies.

c. Sandstone wells - Sixteen municipalities had sandstone wells in 1970, public pumpage was 5.57 mgd. This accounted for 77% of the total withdrawal from sandstone wells in the county. Domestic pumpage from sandstone wells is quite limited, because in most areas satisfactory wells for domestic supplies can be developed from either glacial drift or shallow dolomite aquifers (Reference 21).

Groundwater pumpage in Ogle County from 1971 through 1976 is summarized in Table 2.4-24. As indicated, the groundwater pumpage has been relatively constant at less than 11 mgd, with the exception of 1974 when the pumpage was approximately 13.6 mgd.

Figure 2.4-27 is a contour map of the elevation of the piezometric surface of the Cambrian-Ordovician Aquifer in northern Illinois based on piezometric elevations measured in October 1971 (Reference 22). In the general region of the Byron site, the Rock River and the city of Rockford are the main groundwater discharge areas that control the slope of the piezometric surface. The Byron site is located within a trough in the piezometric surface resulting from discharge from the aquifer to the Rock River.

Yearly changes in piezometric levels at public groundwater wells in the Cambrian-Ordovician Aquifer are shown on Table 2.4-22. These data show very little fluctuation, for the period of record, in the piezometric surface within a 10-mile radius of the Byron site.

2.4.13.2.2 <u>Future Regional Groundwater Use</u>

Groundwater pumpage in Ogle county, public, industrial, and domestic, was projected in 1966 to double in the next 10 to 25 years (References 23 and 24). However, groundwater pumpage in Ogle County has actually decreased at an average rate of 240,000 gpd per year from 1965 to 1970. Groundwater pumpage from 1971 through 1976 is summarized in Table 2.4-24. Due to the relatively low level of urbanization around the site area and the small amount of onsite use, it is unlikely that future increases in groundwater withdrawal in the area would have much effect on the groundwater supply at the site.

2.4.13.2.3 Present Site Groundwater Use

Piezometers and observation wells were installed at the site as part of the groundwater monitoring program. Table 2.4-25 lists the borings which have piezometers, the depth and elevation at which they were installed, and the hydrogeologic unit in which they were installed. Table 2.4-25 also lists the water level elevations measured in the piezometers and observation wells. The locations of the piezometers and observation wells are shown in Figures 2.5-3 and 2.5-5. Groundwater levels presented on the borings logs were recorded during the drilling of the borings.

Piezometer installation included either: (1) a single piezometer in the Galena-Platteville Groups approximately 30 feet below the groundwater level, (2) a double piezometer in which the upper piezometer was installed in the Galena and Platteville Groups approximately 30 feet below the groundwater and a lower piezometer set in the Ancell Group (St. Peter Sandstone) and sealed from the Galena-Platteville Groups by a bentonite plug in the Harmony Hill Shale Member, or (3) a single piezometer set in the Ancell Group and sealed from the Galena and Platteville Groups by a bentonite plug in the Harmony Hill Shale Member.

In addition, three observation wells were drilled at the locations indicated on Figure 2.5-5 to verify the existence of a perched water level.

On a regional basis, the Galena-Platteville dolomites are hydraulically continuous with the lower sandstone units of the Cambrian-Ordovician Aquifer (Reference 19). However, in the vicinity of the Byron site, groundwater in the Galena-Platteville dolomites is perched on the low permeability Harmony Hill Shale Member of the Glenwood Formation. This is demonstrated by the head relationships measured in the onsite piezometers and observation wells. Piezometric levels in piezometers installed below the Harmony Hill Shale Member are at least 686 feet MSL, whereas piezometric levels in piezometers installed above the Harmony Hill Shale Member are at least 745 feet MSL (Table 2.4-25).

A piezometric surface map of the site vicinity, as measured in the Galena-Platteville dolomite, is shown in Figure 2.4-25. As shown on the map, the main plant structures are situated on a recharge area of the Galena-Platteville portion of the aquifer. The aquifer is recharged by direct infiltration of precipitation through the overlying, thin glacial drift. The Galena-Platteville dolomites have little primary permeability and precipitation moves downward from the overlying drift into solution enlarged joints that provide secondary permeability. Groundwater flows radially from the site, but the principal discharge boundaries are the Rock River to the west and northwest of the site and Black Walnut Creek to the east and southeast of the site.

Table 2.4-26 lists recorded active, domestic, or agricultural groundwater wells east of the Rock River with 2.25 miles of the site. These wells are primarily completed in the Galena-Platteville dolomites. Figure 2.4-28 illustrates the location of each well within 2.25 miles from the plant site. Domestic and agricultural wells west of the Rock River are not shown on Figure 2.4-28 because the river is a common discharge boundary for wells, east and west of the river, which are completed in the Galena-Platteville dolomites.

Pump tests were performed on June 20 and July 2, 1974 in two domestic water supply wells that are completed in the Galena-Platteville dolomites. These two wells are located at the western edge of the site along Razorville Road. From the pump tests, aquifer parameters were derived based on July 1, 1974, piezometric levels. Based on estimated saturated thicknesses of 111 feet and 90 feet at these two wells, the hydraulic conductivity of this portion of the aquifer was 3.6 gpd/ft^2 and 222.2 gpd/ft^2 respectively. The effective porosity of this portion of the aquifer is estimated to range from 0.05 to 0.10.

2.4.13.2.4 Future Site Groundwater Use

There are no anticipated changes in the present pattern of groundwater use in the site area.

2.4.13.2.5 Effects of Plant Groundwater Use

The projected effects of plant groundwater withdrawals of approximately 470 gpm have been evaluated using the Theis equations (Reference 25) with assumed values of 17,000 gpd/ft and 3.5×10^{-4} for the coefficients of transmissivity and storage (Reference 18). The projected effects were reevaluated using the values of 40,000 gpd/ft and 2.5×10^{-4} determined from the 1980 aquifer pumping test. Theoretical distance-drawdown and time-drawdown curves were constructed in order to determine the anticipated shape of the cone of depression and the radius of influence of the Byron Station wells. These theoretical curves indicate that the Byron groundwater withdrawals should not impose measurable interference drawdowns on the nearest public water supply wells completed in the Cambrian-Ordovician or Mt. Simon Aquifers. Indeed, the groundwater withdrawals at the Byron site will intercept groundwater that otherwise would naturally discharge from the aquifer into the Rock River.

The effects of pumping from Byron Station water wells will be minimal on domestic wells completed in the Galena-Platteville dolomites. As described in Subsection 2.4.13.2.3, the Galena-Platteville dolomites in the site vicinity are hydraulically separated from the lower portion of the Cambrian-Ordovician Aquifer by the Harmony Hill Shale Member of the Glenwood Formation. In addition, the Byron Station water wells are cased through the Galena-Platteville dolomites and the underlying Ancell Group (St. Peter Sandstone). Groundwater in the Galena-Platteville dolomites is perched on the Harmony Hill Shale Member and initially water levels in this aquifer will not be lowered by pumping for daily plant use from the lower portion of the Cambrian-Ordovician Aquifer and Mt. Simon Aquifer. As pumpage from the plant water wells continues with time, minor vertical leakage may occur through the Harmony Hill Shale Member. If recharge by rainfall infiltration is not considered, water levels in domestic and agricultural wells in the site vicinity may be lowered slightly as a result of long-term pumping of groundwater from the Byron Station water wells. Measurements made during the 1980 aquifer pumping test verified that the offsite drawdown effects in the Galena-Platteville dolomites and Ancell Group (St. Peter Sandstone) will be very minor.

2.4.13.3 Accident Effects

As described in Subsection 2.4.12, the largest tanks located outside the containment building and containing radioactive effluents are the boron recycle holdup tanks. These tanks are located in a portion of the Seismic Category I auxiliary building, where the floor elevation is 815.0 feet. Each of the recycle holdup tanks has a capacity of 125,000 gallons. The design-basis liquid phase radionuclide content of each tank is given in Table 2.4-20.

The plant grade elevation is 869.0 feet. The site area piezometric surface map, as measured on July 1, 1974, showed a groundwater elevation of approximately 840.0 feet. However, taking seasonal variations into account, it is conservatively assumed that the groundwater elevation at the time of the postulated accident would be 808.0 feet. The nearest offsite downgradient groundwater user is located at about 1960 feet from the auxiliary building (Well 55, Figure 2.4-28). The prevailing hydraulic gradient between the auxiliary building and the nearest well is 0.0186. There is a spring located at a distance of about 3630 feet downgradient from the auxiliary building. The hydraulic gradient between the auxiliary building and this spring is 0.011 which is flatter than the gradient to the nearest offsite well. It is, therefore, concluded that the critical path of the movement of accidentally released effluents will be from the auxiliary building to the nearest downgradient offsite well. It is anticipated that there will be no future groundwater user within 1960 feet of the auxiliary building. Therefore, the results of the analysis of accident effects for the existing well are valid for any future wells also.

The rock mass in the vicinity of the auxiliary building was grouted to fill solution features and joints. This would greatly restrict the seepage of any accidentally released effluents into the surrounding groundwater environment. However, for a conservative evaluation of the accident effects, the following analysis was performed.

It is postulated that a 0.1-inch wide crack develops throughout the width of the auxiliary building. At the same time, one of the boron recycle holdup tanks ruptures and spills its entire content over the floor of the building which leaks into the underlying rock mass through the postulated crack. The effluents are postulated to move through the grouted rock mass to the ambient groundwater and then follow a calculated maximum hydraulic gradient of 0.0186, measured between the floor of the auxiliary building foundation mat and the nearest well number 55 (renumbered as well number 7).

The observed hydraulic conductivity of the grouted rock mass directly under the excavation is 2.44×10^{-6} cfs/ft^2. The rate of vertical leakage of effluents through the postulated crack is estimated to be 2.03×10^{-8} cfs per foot length of the crack.

Field assessments of the hydraulic characteristics of site bedrock natural fractures were performed by drilling holes which intersected joints and bedding planes and injecting water under constant pressure between packers set in the bore hole. The hydraulic conductivity of the ungrouted rock was determined from water pressure test data presented in the following borings P2 through P7, P9, P10, P15A, P22, and P23. Previous interpretations of hydraulic conductivity were derived primarily from off-site pump tests, however; as a result of reevaluation of the test methods, brevity of the test, and location of the test site it was concluded plant site data would be more representative of the aquifer characteristics.

The effective porosity of the Galena-Platteville aquifer was estimated from the geophysical logs indicated on the G series borings to range from 0.05 to 0.10. The average saturated thickness of the aquifer based upon the auxiliary building floor mat elevation and the lowest measured water surface of well number 55 (renumbered as well number 7) is 135 feet.

Many phenomena can combine to reduce concentrations of radioactivity released to ground water. These include initial dilution by the receiving body of water, subsequent dilution and dispersion enroute, absorption and ion exchange en route, dilution of waters at a well by other waters drawn into its core of depression, and radioactive decay experienced due to the travel time. To be conservative, the present analysis considers only the effects of initial dilution and radioactive decay. The acceptance criteria are 10 CFR Part 20, Appendix B, concentration limits for liquids in an unrestricted area.

For travel time in excess of 0.5 years, the concentrations of all but three nuclides given in Table 2.4-20 will be reduced by radioactive decay alone to values which are less than 10 CFR 20 limits. The three exceptions are CS-134, CS-137, and H-3 with half-lives of 2.06y, 30.17y and 12.3y respectively; these would require reductions by factors of 2555, 750, and 1167, respectively, to reach 10 CFR 20 limits.

The present analysis results in an initial dilution factor of 2,200 and a transient time of 30.5 years which in configuration are more than sufficient to result in concentrations at the nearest well to be within 10 CFR 20 limits.

2.4.13.4 Monitoring

A site groundwater monitoring program was begun in December 1975. This monitoring program was performed (1) to define existing conditions as a base for future comparisons; (2) to monitor the effects of construction; (3) to check for either plant operation or groundwater use by others; and (4) to protect offsite groundwater users in case of detrimental changes in groundwater quality. The site groundwater monitoring program was not part of any radiological monitoring program.

Six domestic and agricultural water wells were monitored for monthly changes in piezometric levels. Three of the water wells are now owned by Exelon Generation Company and are located on the inside perimeter of the Byron site boundaries. The other three wells are on the outside perimeter of the site boundary. Data from this monitoring program indicated no changes in groundwater chemistry or piezometric levels attributable to excavation, grouting, groundwater pumping or other activities at the Byron site. Groundwater chemistry and water level data will provide the basis for comparison with any future data collected. The groundwater chemistry and water level data are provided in Tables 2.4-27 and 2.4-28, respectively.

In addition to this site groundwater monitoring program, the detailed site geotechnical investigation identified on area groundwater contamination by toxic materials prior to the purchase of the land by Commonwealth Edison. An investigation of the nature and the extent of the contamination was performed (Reference 26).

2.4.13.5 Design Bases for Subsurface Hydrostatic Loading

The design parameters for groundwater induced hydrostatic loadings on subsurface portions of site safety-related structures are based on a piezometric level of 869.0 feet MSL. This design groundwater level corresponds to plant grade.

For the design and implementation of site construction dewatering systems refer to Subsections 2.5.4.5 and 2.5.4.6.

2.4.14 Technical Specification and Emergency Operating Requirements

The limiting conditions for operation based on the water levels and flows in the Rock River near the screen house are discussed in Section 3.7.9 of the Technical Specifications for Byron 1 & 2.

2.4.15 References

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TABLE 2.4-1

DRAINAGE AREAS OF THE ROCK RIVER

LOCATION	DRAINAGE AREA (mi^2)
Illinois-Wisconsin State Line	3,463
Spillway at Rockton	3,718
Rockton Stream-Gauging Station	6,363
Fordam Dam in Rockford	6,535
River Screen House at Byron Station	8,170
Oregon	8,205
Dixon Dam	8,615
Upper Rock Falls Dam	8,746
Sterling	8,749
Como Stream-Gauging Station	8,755
Prophetstown	9,193
Confluence with Mississippi River	10,917

Source: Reference 1

TABLE 2.4-2

DAMS ON THE ROCK RIVER NEAR THE SITE

LOCATION	RIVER MILE ABOVE MISSISSIPPI RIVER	DRAINAGE AREA IN mi^2	PRESENT USE	OWNER
Sterling	72.9	8749	Maintains Water Level	Sterling Park District
Sterling	73.8	8746	Maintains Water Level	Ill. Dept Conser-vation
Dixon	87.3	8615	Generates Power	Common-wealth Edison Company
Oregon	109.7	8205	Maintains Water Level	Ill. Dept Conser-vation
Rockford	136.7	6535	Maintains Water Level	Common-wealth Edison Co.
Rockton	159.1	3718	Generates Power	South Beloit Water, Gas & Electric Co.

Note: River screen house is located at river mile 115.

Source: Reference 27.

TABLE 2.4-3

OWNER AND LOCATION OF SURFACE WATER INTAKES DOWNSTREAM OF THE SITE

OWNER	LOCATION	RATE OF WITHDRAWAL (cfs)	RIVER MILE ABOVE MISSISSIPPI RIVER
Northwestern Steel & Wire Co.	Sterling	41	72.7 Right Bank
Lawrence Brothers Inc.	Sterling	Not available	73.1 Right Bank
Illinois Northern Utilities Co.	Sterling	Not available	76.1 Right Bank
Commonwealth Edison Co. (Hydroelectric Plant)	Dixon	5000 Maximum	87.3 Right Bank
Medusa Cement Co.	Dixon	0.25	89 Right Bank
Reynolds Wire Co.	Dixon	Not available	90.2 Left Bank
Sandusky Cement Co.	Dixon	Not available	93.0 Left Bank

Note: River screen house is located at river mile 115.

Source: Reference 27.

TABLE 2.4-4

FLOOD CREST ELEVATIONS ABOVE 10.0-FOOT

STAGE ON THE ROCK RIVER AT ROCKTON

DATE OF CREST	GAUGE HEIGHT STAGE (ft)	GAUGE HEIGHT ELEV. (ft)	DISCHARGE (cfs)
March 22, 1904	13.80*	722.74	28,500
March 26, 1905	11.56*	720.50	23,000
March 3, 1906	11.30*	720.24	22,300
March 30, 1916	13.06**	-	32,500
March 14, 1918	11.40**	-	24,400
March 19, 1919	11.50**	-	24,800
February, 1937	14.6***	-	-
August 27, 1940	10.53	718.47	15,800
March 16, 1943	12.88	720.82	22,800
February 27, 1944	10.00	717.94	15,000
March 15, 1944	10.91	718.85	17,300
January 9, 1946	12.24	720.18	20,900
March 16, 1946	10.90	718.84	18,700
March 1, 1948	11.80	719.74	19,800
March 20, 1948	13.82	721.76	25,700
March 9, 1949	10.57	718.51	16,500
March 22, 1952	11.82	719.76	19,800
April 4, 1959	14.08	722.02	25,400
January 17, 1960	10.56	718.50	16,300
April 2, 1960	13.49	721.43	23,800
May 11-12,1960	10.78	718.72	16,800
March 29-April 1, 1962	11.32	719.26	18,000
January 24, 1969	10.0	717.94	15,000
July 9, 1969	10.59****	718.53	14,500
February 27, 1971	11.22	719.16	17,800
April 22, 1973	13.40	721.34	24,300
March 11, 1974	11.32	719.26	19,900
March 25, 1975	15.54	723.48	30,000
March 5, 1976	10.45	718.39	16,700
Mean annual flow	5.06	713.00	3,901

* Datum 1 foot higher than present datum at elevation 707.94.

** For gauge at Rockford, datum unknown; discharge adjusted to Rockton.

*** From painted flood mark; discharge not recorded; due to ice jam.

**** Backwater from debris jam.

TABLE 2.4-5

TEN LARGEST RECORDED FLOODS ON

THE ROCK RIVER AT ROCKTON

ORDER NUMBER	DATE OF CREST	GAUGE HEIGHT STAGE (ft.)	GAUGE HEIGHT ELEV. (ft.)	DISCHARGE (ft^3/s)
1	March 30, 1916	13.06*	-	32,500
2	March 25, 1975	15.54	723.48	30,000
3	March 22, 1904	13.80**	722.74	28,500
4	March 20, 1948	13.82	721.76	25,700
5	April 4, 1959	14.08	722.02	25,400
6	March 19, 1919	11.50*	-	24,800
7	March 14, 1918	11.40*	-	24,400
8	April 2, 1960	13.49	721.43	23,800
9	April 22, 1973	13.40	721.34	24,300
10	March 25, 1905	11.56**	720.50	23,000

* For gauge at Rockford, datum unknown, discharge adjusted to Rockton.

** Datum 1 foot higher than present datum at elevation 707.94.

TABLE 2.4-6

PEAK FLOW FREQUENCY FOR THE ROCK RIVER

AT THE SITE AREA

RECURRENCE INTERVAL (yr)	PEAK FLOW (ft^3/s)
5	34,600
10	42,000
25	50,800
50	56,900
100	62,600

TABLE 2.4-7

48-HOUR LOCAL PROBABLE MAXIMUM PRECIPITATION

6-HOUR INCREMENTS

6-HOUR TIME PERIOD	INCREMENTAL RAINFALL (inches)	CUMULATIVE RAINFALL (inches)
1	0.28	0.28
2	0.68	0.96
3	0.91	1.87
4	0.40	2.27
5	1.37	3.64
6	2.27	5.91
7	26.56	32.47
8	1.81	34.38

TABLE 2.4-7a

MAXIMUM RAINFALL INTENSITY DURING LOCAL PROBABLE MAXIMUM PRECIPITATION

TIME	CUMULATIVE RAINFALL (inches)	RAINFALL INTENSITY (inches/hour)
5 minutes	5.91	70.9
15 minutes	9.36	37.4
30 minutes	13.41	26.8
60 minutes	17.6	17.6

TABLE 2.4-7b

CULVERT SCHEDULE

NO.	SIZE AND TYPE	LENGTH (ft)	SLOPE (%)	INLET INVERT EL.	OUTLET INVERT EL.
C2	1-48" C.M.P.*	278.0	0.935	863.60	861.00
C3	2-24" R.C.P., 4-0" O.C.	52.0	0.480	865.25	865.00
C4	1-54" C.M.P.	86.0	0.930	859.40	858.60
C8	1-48" C.M.P.	50.0	0.400	861.90	861.70
C9	1-54" C.M.P.	54.0	0.370	861.00	860.80

*C.M.P. = corrugated metal pipe.
R.C.P. = reinforced concrete pipe.
O.C. = on center.

TABLE 2.4-8

ROCK RIVER BASIN STANDARD PROJECT STORM DISTRIBUTION

SUB-BASIN 1		SUB-BASIN 2		SUB-BASIN 2		SUB-BASIN 4				SUB-BASIN 5		SUB-BASIN 6		SUB-BASIN 7	
TIME (hr)	PRECIP-ITATION (in.)	TIME (hr)	PRECIP-ITATION (in.)	TIME (hr)	PRECIP-ITATION (in.)	TIME (hr)	PRECIP-ITATION (in.)	TIME (hr)	PRECIP-ITATION (in.)	TIME (hr)	PRECIP-ITATION (in.)	TIME (hr)	PRECIP-ITATION (in.)	TIME (hr)	PRECIP-ITATION (in.)
0	0	0	0	0	0	0	0	38	5.38	0	0	0	0	0	0
12	0.18	4	0.03	8	0.10	2	0.02	40	0.58	4	0.03	8	0.11	6	0.04
24	0.32	8	0.04	16	0.19	4	0.02	42	0.30	8	0.04	16	0.18	12	0.08
36	0.66	12	0.08	24	0.28	6	0.03	44	0.20	12	0.08	24	0.26	18	0.15
48	3.20	16	0.08	32	0.56	8	0.03	46	0.20	16	0.09	32	0.50	24	0.26
60	0.44	20	0.16	40	6.51	10	0.03	48	0.10	20	0.12	40	5.31	30	0.37
72	0.21	24	0.20	48	0.94	12	0.04	50	0.10	24	0.17	48	0.72	36	0.73

TABLE 2.4-8 (Cont'd)

SUB-BASIN 1		SUB-BASIN 2		SUB-BASIN 2		SUB-BASIN 4				SUB-BASIN 5		SUB-BASIN 6		SUB-BASIN 7	
TIME (hr)	PRECIP-ITATION (in.)	TIME (hr)	PRECIP-ITATION (in.)	TIME (hr)	PRECIP-ITATION (in.)	TIME (hr)	PRECIP-ITATION (in.)	TIME (hr)	PRECIP-ITATION (in.)	TIME (hr)	PRECIP-ITATION (in.)	TIME (hr)	PRECIP-ITATION (in.)	TIME (hr)	PRECIP-ITATION (in.)
		28	0.24	56	0.38	14	0.04	52	0.10	28	0.30	56	0.35	42	5.70
		32	0.35	64	0.29	16	0.04	54	0.09	32	0.42	64	0.18	48	0.44
		36	0.67	72	0.19	18	0.09	56	0.09	36	0.76	72	0.14	54	0.33
		40	5.80			20	0.09	58	0.09	40	5.96			60	0.18
		44	0.43			22	0.09	60	0.04	44	0.42			66	0.11
		48	0.32			24	0.10	62	0.04	48	0.30			72	0.03
		52	0.24			26	0.10	64	0.03	52	0.26				
		56	0.16			28	0.14	66	0.03	56	0.17				
		60	0.12			30	0.20	68	0.03	60	0.12				
		64	0.08			32	0.30	70	0.02	64	0.08				
		68	0.04			34	0.39	72	0.02	68	0.09				
		72	0			36	0.68			72	0.04				
Total	5.01		9.04		9.44				9.77		9.45		7.75		8.42

TABLE 2.4-9

STANDARD PROJECT FLOOD MINIMUM RETENTION RATES

SUB-BASIN (1)	SOIL GROUP (2)	PROPORTION (3)	MINIMUM* RETENTION (in./hr) (4)	(3) x (4) (in./hr) (5)	WEIGHTED RETENTION (in./hr) (6)
I	B	0.5	0.24	0.12	0.14
	D	0.5	0.04	0.02	
II	A	0.4	0.40	0.16	0.27
	B	0.4	0.24	0.10	
	D	0.2	0.04	0.01	
III	B	0.9	0.24	0.22	0.22
	D	0.1	0.04	0.00	
IV	B	0.9	0.24	0.22	0.22
	D	0.1	0.04	0.00	
V	B	0.9	0.24	0.22	0.22
	D	0.1	0.04	0.00	
VI	B	0.8	0.24	0.19	0.20
	D	0.2	0.04	0.01	
VII	B	0.8	0.24	0.19	0.20
	D	0.2	0.04	0.01	

* Recommended minimum rate for use in general case as given in the table on page 64, "Design of Small Dams," by U.S. Bureau of Reclamation, 1973.

TABLE 2.4-10

ROCK RIVER BASIN CHARACTERISTICS

AND UNIT HYDROGRAPH PARAMETERS

SUB-BASIN	DRAINAGE AREA (mi^2)	L (mi)	L_c (mi)	C_p	C_t	DURATION OF HYDROGRAPH (hr)	PEAK DISCHARGE ($ft^3/s/mi^2$)
I	3,151	151.3	51	-	-	12	2.1
II	529	47.4	16.6	0.54	3.8	4	15.1
III	1,090	66.8	23.3	0.41	2.2	8	13.2
IV	700	37.1	12.2	0.41	2.2	2	19.1
V	753	98.8	39.9	0.41	2.2	4	10.1
VI	1,330	85.1	47.8	0.50	4.4	8	6.2
VII	527	53.7	22.5	0.50	4.4	6	8.8

TABLE 2.4-11

FLOOD DISCHARGE AT BYRON STATION 90% CONFIDENCE VALUES

(Q_{SITE} = 0.96 Q_{COMO} SKEW COEFFICIENT = -0.50)

MEAN RETURN PERIOD YEARS	K	LOGFLOW 10^3 (cfs)	10% ERROR LIMIT IN STANDARD DEVIATION	LOGFLOW FOR 90% CONFIDENCE	90% CONFIDENCE FLOW 10^3 (cfs)
10	1.216	1.623 (42.0)	0.28	1.689	48.9
10^2	1.955	1.797 (62.6)	0.42	1.894	78.4
10^3	2.399	1.900 (79.5)	0.52	2.022	105.2
10^4	2.708	1.973 (94.0)	0.61	2.133	136.1
10^5	2.960	2.032 (107.7)	0.69	2.194	156.2
10^6	3.150	2.073 (118.4)	0.75	2.250	178.0

Mean Logflow (M) = 1.339
Standard Deviation (S) = 0.234
Coefficient of Skewness (g) = -.750

TABLE 2.4-12

FLOOD ELEVATIONS AT INTAKE

FLOW CONDITIONS	FLOW (ft^3/s)	ELEVATION (ft)
Probable maximum flood (PMF)	308,000	708.3
Combined event flood	178,000	698.7
Standard project flood (SPF)	154,000	695.6
Flood of record (FOR)	57,670	683.6
Mean annual flow	4,730	672.0
Lowest 1-day flow	400	670.4

TABLE 2.4-13

FRICTION, CONTRACTION AND EXPANSION COEFFICIENTS

CROSS-SECTION	MANNING'S n
Channel	0.032
Channel islands	0.075
Overbank	0.075
Contraction	0.1
Expansion	0.3*

*See Reference 28

TABLE 2.4-14

WIND WAVES ON ROCK RIVER COINCIDENT

WITH COMBINED EVENT FLOOD LEVEL

Wind speed overland, mph		40
Fetch, mi		2.10
Wind speed over water, mph		45.2
Wind tide:	depth, ft	16.9
	setup, ft	0.18
Wave runup:	effective fetch, mi	1.05
	embankment slope, degree	90
	significant wave runup, ft	2.59
	maximum wave runup, ft	4.53
Total setup plus runup:		
	significant wave, ft	2.77
	maximum wave, ft	4.71
Combined event flood level, ft MSL		698.68
Combined event flood level with setup plus runup:		
	significant wave, ft MSL	701.45
	maximum wave, ft MSL	703.39

TABLE 2.4-15

LOW FLOWS IN THE ROCK RIVER AT THE INTAKE

DURATION IN DAYS	DISCHARGE IN $ft^3/_s$ RECURRENCE INTERVAL IN YEARS					
	100	50	20	10	5	2
1	454	512	612	714	856	1,187
3	566	627	730	835	980	1,323
7	643	707	815	925	1,077	1,436
14	674	741	855	970	1,130	1,510
30	739	813	938	1,064	1,239	1,653
60	830	907	1,038	1,172	1,362	1,829
90	921	999	1,134	1,276	1,481	2,007
120	963	1,050	1,200	1,359	1,591	2,195
183	983	1,091	1,280	1,481	1,776	2,547
365	1,881	2,101	2,473	2,849	3,367	4,575

TABLE 2.4-16

COOLING WATER CAPABILITIES OF VARIOUS PUMPS AND WELLS

SYSTEM	NOMINAL PUMP OR WELL RATED FLOW*	NUMBER OF PUMPS OR WELLS
Circulating water	214,500 gpm/pump	6
Nonessential Service water	35,000 gpm/pump	3
Essential service water makeup	1,500 gpm/pump	2
Circulating water makeup	24,000 gpm/pump	3
Deep wells	800 gpm/well	2

* These flows are for a specific pump operating point. Actual flow rates may vary depending on system lineups and plant operating modes.

TABLE 2.4-17

DATA ON ESSENTIAL SERVICE WATER SYSTEM INTAKE

	INTAKE
Required minimum safety-related cooling water flow (gpm)	1410
Sump invert elevation (ft, MSL)	660.5
Sump configuration	Drawing M-20
Minimum design operating level (ft, MSL)	664.33
Required minimum pump submergence (inches/ft)	22½/1.875

TABLE 2.4-18

EFFECT OF 10% WITHDRAWAL ON ROCK RIVER LEVELS

	RIVER FLOW		
	1-DAY LOWEST	1-DAY 10 YEAR LOW	7-DAY 10-YEAR LOW
River discharge, ft^3/s			
without 10% withdrawal	400	714	925
with 10% withdrawal	360	643	833
Water surface elevation at withdrawal point, ft (MSL)	670.4	670.5	670.6
Average drop in water surface due to withdrawal, ft			
withdrawal point to Oregon Dam	0.02	0.03	0.03
Oregon to Sterling	0.05	0.07	0.09

TABLE 2.4-19

MONTHLY AVERAGE MEAN AND MINIMUM FLOWS OF

ROCK RIVER AT INTAKE FOR PERIOD 1967-1976

MONTH	AVERAGE MEAN FLOWS (ft^3/s)	AVERAGE MINIMUM FLOWS (ft^3/s)
October	3810	2775
November	4167	3046
December	4088	2972
January	4540	2952
February	5180	3177
March	10526	5533
April	10187	7560
May	8960	5836
June	6503	4089
July	4672	3058
August	3170	2206
September	3552	2166

TABLE 2.4-20

INVENTORY OF LIQUID PHASE ISOTOPES IN

RECYCLE HOLDUP TANK

ISOTOPE	ACTIVITY (μCi/g)
R_b-88	3.7×10^{-2}
R_b-89	2.1×10^{-3}
M_o-99	5.3×10^{-2}
I-131	2.5×10^{-2}
I-132	2.8×10^{-2}
I-133	4.0×10^{-2}
I-134	5.6×10^{-3}
I-135	2.2×10^{-2}
C_s-134	2.3×10^{-2}
C_s-136	2.8×10^{-10}
C_s-137	1.5×10^{-2}
C_s-138	9.8×10^{-3}
B_a-137_m	1.4×10^{-2}
H-3	3.5×10^{0}

TABLE 2.4-21

GENERALIZED SITE HYDROGEOLOGIC COLUMN

UNIT	APPROX. DEPTH TO TOP (ft)	APPROX. DEPTH TO BOTTOM (ft)	APPROX. THICK-NESS (ft)	HYDROGEOLOGY
Glacial drift	0	16	16	Not an aquifer
Cambrian-Ordovician Aquifer*	16	915	900	Major aquifer
Galena and Platteville Groups	16	200	190	Minor unit
St. Peter Sandstone	225	675	450	Important unit
Ironton and Galesville Sandstones	805	915	110	Important unit
Eau Claire Formation	915	1320	405	Not an aquifer
Mt. Simon Sandstone	1320	2750	1430	Important unit, salty at depth

*Only the most important units are listed.

TABLE 2.4-22

PULBIC GROUNDWATER SUPPLIES WITHIN 10 MILES

NAME	LOCATION*	DISTANCE FROM SITE (miles)	WELL NUMBER	DATE DRILLED	TOTAL DEPTH (feet/ surface elevation, MSL)	LOWEST HYDRO-STRATI-GRAPHIC UNIT PENETRATED	ELEVATION OF PIEZOMETRIC SURFACE (feet, MSL/ date)**	1975 AVERAGE DAILY USE (gpd)	REMARKS
Leaf River	25N9E-36	7.5	1	1914	125	Silurian Dolomite	659 ('71),	0	abandoned, 1946
	25N9E-36.5d		2	1945	325 (765)	St. Peter	702 ('73)	64,000	
Lowden Memorial State Park	24N10E-34.6b	3.7	1	1950	415	St. Peter	NA***	NA***	
	24N10E-34.4c		2	1963	295	St. Peter	NA	NA	
Mt. Morris	24N9E-26.8c1	8.4	1	1894	500	St. Peter	NA	NA	abandoned, 1923
	24N9E-26.8c2		2	1920	1147	Ironton-Galesville	NA	NA	emergency use only
	24N9E-27.1f1		3	1926	1807	Mt. Simon	640 ('66), 635 ('71)	129,000	
	24N9E-27.1a		4	1950	1452	Mt. Simon	648('66), 627 ('71)	206,000	
Northern Illi-Nois Univ.	24N10E-34.8e	3.5	1	1953	435	St. Peter	667 ('67)	5, 470 (total wells 1 and 2)	
	24N10E-34.7d		2	1970	580	St. Peter	665 ('70)		
Lorado Taft Field Campus									
Oregon	23N10E-3.6d		1	1886	1690 (672)	Mt. Simon	650 ('66), 652 ('71), 642 ('76)	63,000	

TABLE 2.4-22 (Cont'd)

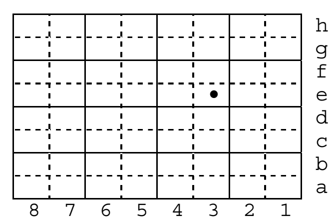
NAME	LOCATION*	DISTANCE FROM SITE (miles)	WELL NUMBER	DATE DRILLED	TOTAL DEPTH (feet/ surface elevation, MSL)	LOWEST HYDRO-STRATI-GRAPHIC UNIT PENETRATED	ELEVATION OF PIEZOMETRIC SURFACE (feet, MSL/ date)**	1975 AVERAGE DAILY USE (gpd)	REMARKS
	23N10E-3.6g		2	1948	1200 (707)	Mt. Simon	661 ('66), 657 ('71), 660 ('76)	166,000	
	23N10E-3.7g		3	1964	1200 (710)	Mt. Simon	660 ('66), 655 ('71), 648 ('76)	207,000	
Stillman Valley	24N11E-1.2b	5.9	1	1938	300 (725)	St. Peter	660 ('71)	17,000	
	24N11E-1.7a		2	1954	445 (740)	St. Peter	660 ('64)	70,000	
Byron	25N11E-32.8e1	4.2	1	1900	2000	Mt. Simon	661 ('68)	210,000	
	25N11E-32.8e2		2	1929	673 (720)	Galesville	659 ('66), 662 ('68)	0	
	25N11E-32.6g		3	1969	715 (720)	Galesville	660 ('69) 672 ('75)	123,000	

Notes:

*Locations of pumping centers are shown on Figure 2.4-25. Locations within each section are based upon the system used by the Illinois State Water Survey illustrated by the grid shown below.

**Potentiometric levels were obtained from Reference 21.

***NA indicates data are not available.



Well located in Sec. 17.3e

TABLE 2.4-23

WATER QUALITY DATA - BYRON STATION WATER WELLS

	December 1974 BYRON WATER WELL 1 (EAST)	July 1980 BYRON WATER WELL 1 (EAST)	December 1974 BYRON WATER WELL 2 (WEST)
Calcium; mg/l as $CaCO_3$	150.0	153	143.0
Magnesium; mg/l as $CaCO_3$	119.0	144	139.0
Sodium; mg/l as Na	4.6	2.6	3.6
Potassium; mg/l as K	--	3.4	--
Total Alkalinity; mg/l as $CaCO_3$	301.0	301	320.0
Sulfate; mg/l as SO_4	5.5	8	5.1
Chloride; mg/l as C1	3.2	<1	2.7
Nitrates; mg/l as NO_3	7.8	<0.01(as N)	<0.1
Silica; mg/l as SiO_2	5.1	9.1	8.1
Total Dissolved Solids; mg/l	96.0	328	582.0
Conductivity; µmohs at 25° C	502.0	600	542.0
Iron; mg/l as Fe	0.30	0.46	0.28
Manganese; mg/l as Mn	0.01	0.02	<0.01
Turbidity; FTU	20.0	--	8.0
pH at 25° C	8.0	7.3	7.6
Total Organic Carbon; mg/l	5.0	--	--
Carbon Dioxide; mg/l as CO_2	8.0	38	30.0

1. Groundwater samples collected December 1974 were analyzed by Aqua Systems Corporation, Chicago, Illinois.

2. Groundwater samples collected July 1980 were analyzed by Aqualab Inc., Streamwood, Illinois; except conductivity, which was measured in the field.

TABLE 2.4-24

GROUNDWATER PUMPAGE, OGLE COUNTY (in mgd)

	1971				1972			
	SAND GRAVEL	DOLOMITE	SANDSTONE	TOTAL	SAND GRAVEL	DOLOMITE	SANDSTONE	TOTAL
Municipal	-	-	5.324	5.324	-	0.001	6.133	6.134
Subdivision	-	-	-	-	-	0.010	-	0.010
Institution	-	0.005	0.021	0.026	-	0.005	0.028	0.033
Industrial	0.205	0.003	1.094	1.302	0.047	0.084	1.052	1.183
Irrigation	-	-	0.020	0.020	-	-	0.050	0.050
Domestic	0.384	0.921	0.230	1.535	0.384	0.921	0.230	1.535
Livestock	0.497	1.195	0.298	1.990	0.497	1.195	0.298	1.990
	1.086	2.124	6.987	10.197	0.928	2.216	7.791	10.935

	1973				1974			
	SAND GRAVEL	DOLOMITE	SANDSTONE	TOTAL	SAND GRAVEL	DOLOMITE	SANDSTONE	TOTAL
Municipal	-	0.001	6.155	6.156	-	0.001	8.800	8.801
Subdivision	-	0.028	0.008	0.036	0.002	0.030	0.008	0.040
Institution	-	0.005	0.038	0.043	-	0.005	0.038	0.043
Industrial	0.066	0.003	1.090	1.159	0.020	0.003	1.117	1.140
Irrigation	-	-	0.050	0.050	-	-	0.050	0.050
Domestic	0.384	0.921	0.230	1.535	0.384	0.921	0.230	1.535
Livestock	0.497	1.195	0.298	1.990	0.497	1.195	0.298	1.990
	0.947	2.153	7.869	10.969	0.903	2.155	10.541	13.599

TABLE 2.4-24 (Cont'd)

	1975				1976			
	SAND GRAVEL	DOLOMITE	SANDSTONE	TOTAL	SAND GRAVEL	DOLOMITE	SANDSTONE	TOTAL
Municipal	-	-	5.943	5.943	-	-	6.051	6.051
Subdivision	0.004	0.032	0.010	0.046	0.004	0.032	0.018	0.054
Institution	-	0.005	0.042	0.047	-	0.005	0.042	0.047
Industrial	0.016	-	0.963	0.979	-	-	1.098	1.098
Irrigation	-	-	0.050	0.050	-	-	0.050	0.050
Domestic	0.384	0.921	0.230	1.535	0.384	0.921	0.230	1.535
Livestock	0.497	1.195	0.298	1.990	0.497	1.195	0.298	1.990
	0.901	2.153	7.536	10.590	0.885	2.153	7.787	10.825

Reference: R. T. Sasman, Written Communication, Illinois State Water Survey, Warrenville, Illinois, 1977.

TABLE 2.4-25

SUMMARY OF PIEZOMETER INSTALLATIONS AND

GROUNDWATER MEASUREMENTS

BORING	GROUND SURFACE ELEVATION (ft, MSL)	DEPTH OF PIEZOMETER (ft)	GROUP IN WHICH* PIEZOMETER INSTALLED	DATE OF WATER LEVEL MEASUREMENT	WATER LEVEL ELEVATION (ft, MSL)
G-1	837.6	105	P	11/8,9/72	747.3
				1/25/73	745.7
				6/27/74	751.2
				7/1/74	750.9
		195	A	11/8,9/72	686.2
				1/25/73	686.6
				6/27/74	Dry
				7/1/74	Dry
G-2	847.9	118	P	11/8,9/72	762.6
				1/25/73	760.9
				6/4/74	789.5
				6/17/74	789.5
				6/26/74	789.8
				7/1/74	789.9
				10/16/74	789.4
				10/30/74	790.4
				12/3/74	790.4
				1/6/75	789.9
				4/9/75	789.5

*G = Galena Group
P = Platteville Group
A = Ancell Group (St. Peter Sandstone)

TABLE 2.4-25 (Cont'd)

BORING	GROUND SURFACE ELEVATION (ft, MSL)	DEPTH OF PIEZOMETER (ft)	GROUP IN WHICH* PIEZOMETER INSTALLED	DATE OF WATER LEVEL MEASUREMENT	WATER LEVEL ELEVATION (ft, MSL)
G-3	855.3	63	G	11/8,9/72	802.5
				1/25/73	809.3
				6/27/74	820.6
				7/1/74	821.3
				4/14/75	800.9
		90	P	11/8,9/72	776.1
				1/25/73	776.3
				6/27/74	788.0
				7/1/74	788.8
				4/14/75	769.7
G-4	828.7	95	G	11/8,9/72	782.8
				1/25/73	782.4
				6/17/74	809.2
				6/26/74	809.8
				7/1/74	810.4
G-5	869.0	113	G	11/8,9/72	775.3
				1/25/73	776.5
				6/27/74	809.8
				7/1/74	810.4
				4/14/75	772.8
G-7	865.9	95	G	11/8,9/72	793.5
				1/25/73	774.8
G-8	831.3	120	P	11/8,9/72	769.0
				1/25/73	767.3
				6/17/74	776.5

*G = Galena Group
P = Platteville Group
A = Ancell Group (St. Peter Sandstone)

TABLE 2.4-25 (Cont'd)

BORING	GROUND SURFACE ELEVATION (ft, MSL)	DEPTH OF PIEZOMETER (ft)	GROUP IN WHICH* PIEZOMETER INSTALLED	DATE OF WATER LEVEL MEASUREMENT	WATER LEVEL ELEVATION (ft, MSL)
				6/26/74	774.5
				7/1/74	774.5
		231	A	11/8,9/72	688.3
				1/25/73	688.4
				6/17/74	691.3
				6/26/74	690.0
				7/1/74	691.1
				4/14/75	688.6
G-10	884.3	110	G	1/25/73	840.8
		279	A	11/8,9/72	691.9
				1/25/73	702.6
				6/26/74	693.0
				7/1/74	693.6
G-12	852.8	120	P	11/8,9/72	781.7
				1/25/73	774.8
				6/27/74	807.0
				7/1/74	808.3
G-13	860.43		G	7/1/74	826.4
G-14	796.8	75	P	11/8,9/74	751.5
		181	A	11/8,9/74	686.6
G-15	782.3	116	P	11/8,9/74	719.3
		175	A	11/8,9/74	700.8
G-16	832.2	100	P	11/8,9/74	762.6
		220	A	11/8,9/74	691.9

*G = Galena Group
P = Platteville Group
A = Ancell Group (St. Peter Sandstone)

TABLE 2.4-25 (Cont'd)

BORING	GROUND SURFACE ELEVATION (ft, MSL)	DEPTH OF PIEZOMETER (ft)	GROUP IN WHICH* PIEZOMETER INSTALLED	DATE OF WATER LEVEL MEASUREMENT	WATER LEVEL ELEVATION (ft, MSL)
G-17	840.7	107	P	11/8,9/72	751.8
				1/25/73	752.0
				6/27/74	758.1
				7/1/74	758.3
		249	A	11/8,9/72	686.8
				1/25/73	686.7
				6/27/74	689.3
				7/1/74	689.1
G-18	852.1	120	P	11/8,9/72	760.0
				1/25/73	760.8
				6/27/74	770.6
				7/1/74	771.2
				4/14/75	755.4
		250	A	11/8,9/72	688.4
				1/25/73	688.7
				6/27/74	690.3
				7/1/74	690.8
				4/14/75	688.4
G-19	863.9	111	P	1/25/73	843.6
				6/28/74	826.5
				7/1/74	826.9
G-20	861.1	120	P	11/8,9/72	776.4
				1/25/73	777.1
				6/28/74	772.4
		248	A	11/8,9/72	689.8

*G = Galena Group
P = Platteville Group
A = Ancell Group (St. Peter Sandstone)

TABLE 2.4-25 (Cont'd)

BORING	GROUND SURFACE ELEVATION (ft, MSL)	DEPTH OF PIEZOMETER (ft)	GROUP IN WHICH* PIEZOMETER INSTALLED	DATE OF WATER LEVEL MEASUREMENT	WATER LEVEL ELEVATION (ft, MSL)
				1/25/73	690.1
				6/28/74	671.4
				4/14/75	689.3
G-21	869.5	118	P	11/8,9/72	773.1
				1/25/73	773.4
G-22	855.7	120	P	1/25/73	774.7
				6/17/74	787.7
				6/26/74	786.7
				7/1/74	786.9
				4/14/75	765.6
G-23	676.5			11/8,9/72	672.5
G-24	878.4	123	P	11/8,9/72	794.5
				1/25/73	796.8
		295	A	11/8,9/72	692.6
				1/25/73	695.0
G-25	860.1	119	P	11/8,9/72	790.5
				1/25/73	791.2
		275	A	11/8,9/72	691.4
				1/25/73	692.1
				6/27/74	693.9
				7/1/74	693.9
P-4	881.6	100	G	1/25/73	799.6
P-5	872.8	107	P	1/25/73	803.8
				7/1/74	828.3
				4/14/75	787.5

*G = Galena Group
P = Platteville Group
A = Ancell Group (St. Peter Sandstone)

TABLE 2.4-25 (Cont'd)

BORING	GROUND SURFACE ELEVATION (ft, MSL)	DEPTH OF PIEZOMETER (ft)	GROUP IN WHICH* PIEZOMETER INSTALLED	DATE OF WATER LEVEL MEASUREMENT	WATER LEVEL ELEVATION (ft, MSL)
P-8	874.1	260	A	1/25/73	694.3
P-9	861.3	248	A	1/25/73	692.4
P-10	866.1	104	P	1/25/73	793.1
P-11	867.1	88	G	1/25/73	823.8
P-22	877.3	95	G	1/25/73	798.4
P-39	871.9	100	P	11/29/73	824.9
				7/1/74	845.6
				4/14/75	787.5
0-1	872.1	85	G	1/25/73	817.1
0-2	878.9	85	G	1/25/73	815.2
0-3	878.0	85	G	1/25/73	818.4

*G = Galena Group
P = Platteville Group
A = Ancell Group (St. Peter Sandstone)
Observation wells 0-1, 0-2 and 0-3 were drilled to the depths listed and left open.

TABLE 2.4-26

WELLS WITHIN 2.25 MILES OF PLANT SITE

SECTION	WELL NUMBER	LOCATION* (miles)	DEPTH (ft)	APPROX. PUMPAGE (gpd)	USAGE	OWNER
1	1	N.2-1/2	90	100	Domestic	R. Weems
1	2	N.2-1/4	90	200	Domestic Agricultural	D. McKiskr
1	3	N.2-1/4	--	---	---	Gutzmer
1	4	N.2	unknown			
6	5	N.NE.2	284	100	Domestic	D. Green
6	6	N.NE.2	385	Not in use	Domestic	R. Henricks
6	7	N.NE.2	unknown	100	Domestic	G. Ashby
6	8	N.NE.2	290	100	Domestic	T. Wilson
6	9	N.NE.2	245	100	Domestic	B. Tubbs
6	10	N.NE.2	350	200	Domestic Agricultural	T. Zimmerman
6	11	N.NE.2	---	---	Domestic	

*Location determined from center of Section 13.

TABLE 2.4-26 (Cont'd)

SECTION	WELL NUMBER	LOCATION* (miles)	DEPTH (ft)	APPROX. PUMPAGE (gpd)	USAGE	OWNER
7	12	N.NE.1-1/4	200	300	Domestic Agricultural	Walker
7	13	N.NE.1-1/2	120	250	Domestic Agricultural	G. Reeverts
7	14	N.NE.1-1/2	---	---	---	---
7	15	N.NE.1-3/4	280	100	Domestic	C. Babbitt
7	16	N.NE.1-3/4	100	300	Domestic Agricultural	K. Reeverts
7	17	N.NE.1-1/2	---	---	---	---
7	18	N.NE.1-1/2	unknown	200	Domestic Agricultural	Oltman
7	19	N.NE.1-1/2	135	200	Domestic Agricultural	Oltman
8	20	NE.2-1/4	200	2000	Domestic Agricultural	E. Seabold
8	21	NE.2-1/4	130	600	Domestic Agricultural	R. Stukenberg

*Location determined from center of Section 13.

TABLE 2.4-26 (Cont'd)

SECTION	WELL NUMBER	LOCATION* (miles)	DEPTH (ft)	APPROX. PUMPAGE (gpd)	USAGE	OWNER
8	22	NE.2-1/4	85	500	Domestic Agricultural	S. Case
11	Total of 110 to 115	NW.1-3/4	---	---	Domestic	Rock River Terrace Unincorp.
11	23	NW.1-1/4	109	150	Domestic Agricultural	Kinyon
11	24	NW.1-1/4	unknown	100	Domestic Agricultural	R. Stukenberg
11	25	NW.1	---	---	Domestic	V. Eakle, Jr.
11	26	NW.1	unknown	200	---	Bonties
12	27	N.1-1/4	---	---	Not in use	
12	28	N.1	---	---	Domestic	Rapp
12	29	N.1	84	500	Domestic Agricultural	Palmer
12	30	N.1/2	---	---	---	---
12	31	N.1	---	---	---	R. Wayne

*Location determined from center of Section 13.

TABLE 2.4-26 (Cont'd)

SECTION	WELL NUMBER	LOCATION* (miles)	DEPTH (ft)	APPROX. PUMPAGE (gpd)	USAGE	OWNER
14	32	W.1	---	---	Not in use	---
14	33	W.3/4	135	60	Domestic	Dauenbaugh
14	34	W.1/2	---	---	---	---
14	35	W.1/2	---	---	---	Rapp
14	36	W.1/2	---	---	---	---
15	37	W.2	243	100	Domestic	Landis
15	38	W.2	---	---	---	---
15	39	W.2	---	---	Domestic Agricultural	---
15	40	W.2	---	---	Domestic Agricultural	---
15	41	W.2	90	---	Not in use	Pulver
15	42	W.2	200	100	Domestic	Eklund
15	43	W.2	90	200	Domestic Agricultural	Frye

*Location determined from center of Section 13.

TABLE 2.4-26 (Cont'd)

SECTION	WELL NUMBER	LOCATION* (miles)	DEPTH (ft)	APPROX. PUMPAGE (gpd)	USAGE	OWNER
17	44	E.1-1/2	unknown	100	Domestic	H. Hillman
17	45	E.1-1/2	unknown	100	Domestic	K. Stout
17	46	E.1-3/4	140	200	Domestic Agricultural	R. Case
17	47	E.1-1/2	235	50	Domestic	L. Powell
17	48	E.1-1/2	100	400	Domestic Agricultural	Powell
18	49	E.3/4	310	100	Domestic	F. Seabold
18	50	E.1/2	106	199	Domestic	A. Johnson
18	51	E.1	323	200	Domestic Agricultural	R. Seabold
18	52	E.1-1/2	unknown	200	Domestic Agricultural	A. Bennett
18	53	E.1-1/2	unknown	1000	Domestic Agricultural	J. Anderson
18	54	E.1-1/2	---	---	---	---

*Location determined from center of Section 13.

TABLE 2.4-26 (Cont'd)

SECTION	WELL NUMBER	LOCATION* (miles)	DEPTH (ft)	APPROX. PUMPAGE (gpd)	USAGE	OWNER
19	55	SE.1/2	unknown	100	Domestic	K. Greffe
19	56	SE.1	unknown	150	Domestic	D. Hardesty
19	57	SE.1	unknown	10	Domestic	D. Hardesty
19	58	SE.1	unknown	10	Domestic	D. Hardesty
19	59	SE.1-1/2	---	---	---	---
19	60	SE.1-1/2	---	---	---	---
19	61	SE.1-3/4	unknown	150	Domestic	D. Beemer
19	62	SE.2-1/4	unknown	300	Domestic Agricultural	C. Hepfer
20	63	SE.2	unknown	100	Domestic	M. Bennett
20	64	SE.2-1/4	165	500	Domestic Agricultural	T. O'Hara
22	65	SW.2-1/4	unknown	unknown	Domestic	Mueller
22	66	SW.2-1/4	unknown	100	Domestic	C. Fant
23	67	SW.1-3/4	---	---	Domestic	Devries

*Location determined from center of Section 13.

TABLE 2.4-26 (Cont'd)

SECTION	WELL NUMBER	LOCATION* (miles)	DEPTH (ft)	APPROX. PUMPAGE (gpd)	USAGE	OWNER
23	68	SW.2	---	---	Domestic	Wolfe
23	69	SW.1-1/4	unknown	100	Domestic	Exelon
24	70	S.3/4	200	150	Domestic	J. Devries
24	71	S.1	120	100	Domestic	Becker
24	72	S.1	---	---	---	---
24	73	S.1	95	100	Domestic	Devries
24	74	S.1	---	---	---	---
24	75	S.1-1/4	---	---	Not in use	---
24	76	S.1-1/4	280	100	Domestic	L. Rapoport
24	77	S.1-1/4	60	60	Domestic	A. Greenfield
24	78	S.1-1/4	unknown	100	Domestic	Sherman
24	79	S.1	unknown	100	Domestic	E. Merrell
24	80	S.1	328	100	Domestic	E. Winterton

*Location determined from center of Section 13.

TABLE 2.4-26 (Cont'd)

SECTION	WELL NUMBER	LOCATION* (miles)	DEPTH (ft)	APPROX. PUMPAGE (gpd)	USAGE	OWNER
24	81	S.1-1/2	unknown	50	Domestic	I. Maas
24	82A	S.1-1/2	180	50	Domestic	I. Maas
25	82	S.1-3/4	unknown	10	Domestic	Ebenezer Reformed Church
25	83	S.1-3/4	166	250	Domestic Agricultural	H. Ehmen
25	84	S.1-3/4	285	100	Domestic	A. Gale
25	85	S.2	110	100	Domestic	Jorden
25	86	S.2	90	---	Not in use	J. Yosd
26	87	S.SW.1-3/4	220	400	Domestic Agricultural	R. Bettner
26	88	S.SW.2-1/4	unknown	150	Domestic Agricultural	L. Rushford
30	89	S.SE.2	unknown	10	Domestic	Ebenezer Reformed Church
30	90	S.SE.2	unknown	300	Domestic Agricultural	Blumever

*Location determined from center of Section 13.

TABLE 2.4-26 (Cont'd)

SECTION	WELL NUMBER	LOCATION* (miles)	DEPTH (ft)	APPROX. PUMPAGE (gpd)	USAGE	OWNER
30	91	S.SE.2-1/4	225	100	Domestic	Sheffler
30	92	S.SE.2-1/4	75	100	Domestic	Sheffler

*Location determined from center of Section 13.

TABLE 2.4-27

WATER QUALITY DATA

GALENA-PLATTEVILLE DOLOMITES

WELL 1

DATE	pH	SULPHATE	TOTAL DISSOLVED SOLIDS	TOTAL ALKALINITY	TOTAL HARDNESS	CALCIUM	MAGNESIUM	SOLUBLE IRON	TOTAL* IRON	BORON	CHLORIDE	TOTAL SUSPENDED SOLIDS	OIL AND GREASE
12/9/75	7.3	<10	439	280	315	54	44	0.32	33.0	<0.2	X	X	X
1/20/76	7.2	25	302	228	276	54	35	0.14	15.8	<0.2	7	X	X
2/17/76	7.5	40	413	264	324	56	45	0.06	18.1	<0.2	16	110	X
3/23/76	7.3	33	432	336	456	77	63	0.03	11.7	0.7	9	45	X
4/13/76	8.0	33	426	259	312	63	40	<0.02	4.1	<0.2	8	28	X
5/21/76	7.9	35	422	272	310	65	35	<0.02	1.31	0.6	9	6	X
6/7/76	7.8	40	396	284	305	62	36	0.03	20.6	0.6	8	9	X
7/5/76	7.4	39	510	276	343	69	44	<0.02	0.80	1.3	15	12	<1
8/11/76	7.5	36	420	284	311	56	42	<0.02	6.60	0.8	11	210	<1
9/20/76	7.7	35	400	264	328	63	41	<0.02	3.43	1.6	12	105	<1
10/19/76	7.5	31	378	269	358	58	52	<0.02	3.07	3.6	11	122	<1
11/8/76	7.4	34	394	272	366	73	45	0.04	5.0	2.7	11	237	<1

TABLE 2.4-27 (Cont'd)

WELL 1

DATE	pH	SULPHATE	TOTAL DISSOLVED SOLIDS	TOTAL ALKALINITY	TOTAL HARDNESS	CALCIUM	MAGNESIUM	SOLUBLE IRON	TOTAL* IRON	BORON	CHLORIDE	TOTAL SUSPENDED SOLIDS	OIL AND GREASE
12/6/76	7.4	43	408	264	368	71	46	0.02	5.36	8.0	11	226	2
1/18/77	Not Sampled												
2/22/77	7.4	35	406	268	368	79	42	<0.05	1.79	1.8	13	85	<1
3/21/77	7.9	55	466	310	360	79	39	<0.05	3.58	<0.2	13	150	X
4/26/77	7.4	39	412	252	344	75	38	<0.01	2.85	<0.2	13	98	6
5/10/77	7.6	45	420	252	356	77	40	0.04	1.37	<0.2	13	64	8
6/13/77	7.5	54	404	266	361	72	44	<0.02	1.36	<0.2	12	294	10
7/19/77	Not Sampled												
8/9/77	7.5	60	412	252	340	69	42	<0.02	2.76	<0.2	10	76	6
9/26/77	Not	Sampled											
10/24/77	7.6	28	398	258	333	97	22	0.10	0.43	<0.2	8	23	4
11/28/77	7.8	52	382	250	317	74	32	<0.02	<0.05	<0.2	5	33	5
12/19/77	7.9	37	416	236	312	72	32	<0.05	0.34	<0.2	6	45	1
1/30/78	7.9	36	358	240	288	64	31	<0.05	1.68	<0.2	9	57	1
2/21/78	8.2	31	352	210	286	64	31	<0.05	1.21	2.3	11	35	4

TABLE 2.4-27 (Cont'd)

WELL 1

DATE	pH	SULPHATE	TOTAL DISSOLVED SOLIDS	TOTAL ALKALINITY	TOTAL HARDNESS	CALCIUM	MAGNESIUM	SOLUBLE IRON	TOTAL* IRON	BORON	CHLORIDE	TOTAL SUSPENDED SOLIDS	OIL AND GREASE
3/20/78	7.4	37	382	216	316	61	40	<0.05	1.46	0.3	7	50	4
4/10/78	7.7	33	364	254	350	69	43	0.06	0.62	1.5	9	35	3
5/22/78	8.3	37	416	324	352	75	40	<0.05	14.1	<0.2	9	53	4
6/13/78	7.5	34	390	290	336	72	38	<0.05	0.85	<0.2	8	35	8
8/1/78	Not Sampled												
8/29/78	7.5	37	468	244	304	67	33	<0.05	0.83	<0.2	8	38	1
9/18/78	Not Sampled												
10/16/78	7.6	32	408	284	314	63	38	<0.05	0.42	0.2	X	14	1
11/9/78	7.7	36	388	276	328	64	41	<0.05	0.18	<0.2	9	7	1
12/6/78	7.6	32	386	258	310	63	37	<0.05	0.47	<0.2	11	32	1

*Major source of total iron is the rust from metal well piping and pumping equipment.

NOTES:
1. Well locations are shown on Figure 2.4-28.
2. Samples collected and analyzed by Commonwealth Edison Company.
3. Units for all constituents except pH in milligrams per liter.
4. An X indicates that the sample was not tested for this constituent.

TABLE 2.4-27 (Cont'd)

WELL 2

DATE	pH	SULPHATE	TOTAL DISSOLVED SOLIDS	TOTAL ALKALINITY	TOTAL HARDNESS	CALCIUM	MAGNESIUM	SOLUBLE IRON	TOTAL* IRON	BORON	CHLORIDE	TOTAL SUSPENDED SOLIDS
12/9/75	7.0	50	636	360	495	90	66	0.17	20	<0.2		
1/20/76	Not Sampled											
2/17/76	7.2	100	688	388	536	101	69	<0.02	11.2	<0.2	72	128
3/23/76	7.9	23	385	234	342	68	41	0.03	4.60	0.3	9	25
4/13/76	7.4	35	494	331	422	80	54	<0.02	10.0	<0.2	11	50
5/21/76	7.5	50	540	356	441	84	55	<0.02	5.2	<0.2	13	92
6/7/76	7.4	48	528	346	438	84	56	<0.02	1.4	<0.2	10	12

Sampling of Well 2 discontinued.

*Major source of total iron is the rust from metal well piping and pumping equipment.

NOTES:
1. Well locations are shown in Figure 2.4-28.
2. Samples collected and analyzed by Commonwealth Edison Company.
3. Units for all constituents except pH in milligrams per liter.
4. An X indicates that the sample was not tested for this constituent.

TABLE 2.4-27 (Cont'd)

WELL 3

DATE	pH	SULPHATE	TOTAL DISSOLVED SOLIDS	TOTAL ALKALINITY	TOTAL HARDNESS	CALCIUM	MAGNESIUM	SOLUBLE IRON	TOTAL* IRON	BORON	CHLORIDE	TOTAL SUSPENDED SOLIDS	OIL AND GREASE
12/9/75	7.0	113	1292	355	640	120	83	0.38	24.0	0.2	X	X	X
1/20/76	7.1	125	977	400	548	108	68	0.83	8.0	1.5	61	X	X
2/17/76	7.8	163	1036	460	624	110	85	0.03	8.0	3.2	152	69	X
3/23/76	7.0	100	987	488	688	128	88	0.05	2.40	<0.2	76	28	X
4/13/76	7.2	145	1390	479	666	122	88	0.04	2.00	2.9	77	25	X
5/21/76	7.4	165	1374	496	674	127	86	<0.02	1.45	0.9	76	21	X
6/7/76	7.2	158	1414	532	660	127	83	0.04	3.07	4.1	74	28	X
7/5/76	7.1	158	1435	492	670	122	88	<0.02	1.28	1.0	82	23	<1
8/11/76	7.0	153	1458	488	684	141	80	<0.02	1.90	5.1	76	62	1
9/20/76	7.2	132	1286	498	674	124	88	<0.02	1.64	5.8	79	36	<1
10/19/76	7.1	100	1268	490	685	132	86	<0.02	2.54	14.2	83	16	<1
11/8/76	7.0	148	1270	468	664	128	84	<0.02	2.06	8.0	74	69	<1
12/6/76	7.1	143	1230	466	653	150	68	0.10	2.10	4.3	78	48	1
1/18/77	7.0	148	1338	476	652	150	67	0.08	4.54	5.0	66	150	1
2/22/77	7.1	155	1436	484	675	132	84	<0.05	3.29	7.3	83	159	<1

TABLE 2.4-27 (Cont'd)

WELL 3

DATE	pH	SULPHATE	TOTAL DISSOLVED SOLIDS	TOTAL ALKALINITY	TOTAL HARDNESS	CALCIUM	MAGNESIUM	SOLUBLE IRON	TOTAL* IRON	BORON	CHLORIDE	TOTAL SUSPENDED SOLIDS	OIL AND GREASE
3/21/77	7.3	170	1290	470	675	150	73	0.05	1.71	7.0	85	76	X
4/26/77	7.4	126	1340	484	620	109	85	<0.1	1.18	1.2	78	69	5
5/10/77	7.3	150	1354	516	700	117	99	0.09	2.84	1.5	81	143	5
6/13/77	7.2	155	1308	488	696	141	84	<0.02	2.90	0.4	75	175	12
7/19/77	7.2	148	1388	468	670	133	82	<0.08	2.17	0.2	78	88	4
8/9/77	7.0	110	1412	492	702	131	91	<0.02	0.87	<0.2	81	34	10
9/26/77	7.1	120	1416	488	670	157	68	<0.02	0.03	5.7	65	51	3
10/24/77	7.2	81	1340	490	669	140	77	<0.05	0.25	1.8	64	18	6
11/28/77	7.1	155	1310	476	680	126	89	<0.02	1.82	3.0	78	108	4
12/19/77	7.2	173	1364	480	660	138	77	<0.05	0.41	<0.2	78	16	<1
1/30/78	8.2	151	1272	512	656	133	78	<0.05	0.58	<0.2	85	39	3
2/21/78	7.2	78	1276	490	582	125	66	<0.05	0.30	1.2	86	22	5
3/20/78	7.0	162	1268	496	708	141	87	<0.05	0.59	15.9	83	30	3
4/10/78	Not Sampled												

TABLE 2.4-27 (Cont'd)

WELL 3

DATE	pH	SULPHATE	TOTAL DISSOLVED SOLIDS	TOTAL ALKALINITY	TOTAL HARDNESS	CALCIUM	MAGNESIUM	SOLUBLE IRON	TOTAL* IRON	BORON	CHLORIDE	TOTAL SUSPENDED SOLIDS	OIL AND GREASE
5/22/78	7.1	151	1238	528	658	142	73	<0.05	2.7	<0.2	75	124	6
6/13/78	7.0	143	1172	348	380	91	62	<0.05	1.11	<0.2	30	143	17
8/1/78	7.4	133	1138	516	640	123	81	<0.05	1.12	<0.2	47	26	<1
8/29/78	7.2	130	1214	480	672	136	81	<0.05	0.46	<0.2	80	36	<1
9/18/78	7.2	148	1488	494	662	125	85	0.05	0.72	<0.2	57	59	1
10/16/78	7.1	131	1172	504	1212	133	214	<0.05	0.07	<0.2	X	14	<1
11/9/78	7.0	143	1148	525	690	144	80	<0.05	0.16	<0.2	5	36	1
12/6/78	7.1	50	1200	510	690	132	88	<0.05	<0.05	<0.2	68	8	1

*Major source of total iron is the rust from metal well piping and pumping equipment.

NOTES:
1. Well locations are shown in Figure 2.4-28.
2. Samples collected and analyzed by Commonwealth Edison Company.
3. Units for all constituents except pH in milligrams per liter.
4. An X indicates that the sample was not tested for this constituent.

TABLE 2.4-27 (Cont'd)

WELL 5

DATE	pH	SULPHATE	TOTAL DISSOLVED SOLIDS	TOTAL ALKALINITY	TOTAL HARDNESS	CALCIUM	MAGNESIUM	SOLUBLE IRON	TOTAL* IRON	BORON	CHLORIDE	TOTAL SUSPENDED SOLIDS	OIL AND GREASE
12/9/75	7.4	13	495	400	375	66	51	0.20	28.0	0.2	X	X	X
1/20/76	7.2	22	308	238	312	60	39	0.29	18.9	<0.2	11	X	X
2/17/76	7.4	30	518	336	440	85	55	<0.02	13.4	<0.2	32	104	X
3/23/76	7.2	28	460	332	512	85	72	0.03	4.78	0.3	12	26	X
4/13/76	7.3	33	534	323	440	86	55	0.03	10.5	0.3	15	66	X
5/21/76	7.6	53	588	338	450	88	55	<0.02	8.8	<0.2	19	69	X
6/7/76	7.4	62	518	326	435	84	55	0.09	11.6	0.5	19	67	X
7/5/76	7.4	44	634	284	388	71	51	<0.02	2.01	1.2	13	18	2
8/11/76	7.3	41	508	324	415	86	22	<0.02	5.1	1.3	12	45	1
9/20/76	7.4	42	500	334	409	86	47	<0.02	5.95	1.8	17	36	<1
10/19/76	Not Sampled												
11/8/76	7.1	42	528	350	485	98	56	<0.02	5.70	1.2	17	158	<1
12/6/76	7.3	45	524	336	453	98	51	<0.02	6.35	1.6	23	155	1
1/18/77	7.3	52	452	332	456	90	56	0.07	6.2	0.2	26	63	1
2/22/77	7.3	56	550	326	488	92	63	0.10	5.40	1.4	26	121	1

TABLE 2.4-27 (Cont'd)

WELL 5

DATE	pH	SULPHATE	TOTAL DISSOLVED SOLIDS	TOTAL ALKALINITY	TOTAL HARDNESS	CALCIUM	MAGNESIUM	SOLUBLE IRON	TOTAL* IRON	BORON	CHLORIDE	TOTAL SUSPENDED SOLIDS	OIL AND GREASE
3/21/77	7.5	78	492	330	450	75	64	<0.05	4.78	<0.2	44	136	X
4/26/77	7.1	56	532	320	360	85	52	<0.1	6.80	0.5	19	120	5
5/10/77	7.5	65	574	328	442	92	52	0.04	2.52	<0.2	20	5	2
6/13/77	7.5	115	440	310	403	78	50	<0.02	1.26	<0.2	13	98	4
7/19/77	7.4	52	562	300	400	88	44	<0.02	0.02	<0.2	13	2	1
8/9/77	7.3	80	476	360	398	90	42	<0.02	1.84	<0.2	17	25	7
9/26/77	7.3	49	542	360	294	116	50	<0.02	0.35	<0.2	15	44	1
10/24/77	7.2	40	520	358	498	104	58	<0.05	0.34	<0.2	17	9	1
11/28/77	7.3	55	522	368	494	113	52	<0.02	2.10	<0.2	16	80	9
12/19/77	7.3	48	542	360	480	102	54	<0.05	1.06	<0.2	18	41	12
1/30/78	Not Sampled												
2/21/78	Not Sampled												
3/20/78	Not Sampled												
4/10/78	7.2	46	514	360	510	99	64	0.05	1.82	1.6	20	65	1

TABLE 2.4-27 (Cont'd)

WELL 5

DATE	pH	SULPHATE	TOTAL DISSOLVED SOLIDS	TOTAL ALKALINITY	TOTAL HARDNESS	CALCIUM	MAGNESIUM	SOLUBLE IRON	TOTAL* IRON	BORON	CHLORIDE	TOTAL SUSPENDED SOLIDS	OIL AND GREASE
5/22/78	7.2	65	606	380	520	98	67	<0.05	4.25	0.2	24	38	5
6/13/78	7.2	73	566	502	472	115	93	<0.05	6.07	<0.2	61	118	7
8/1/78	Not Sampled												
8/29/78	7.2	130	1148	360	444	110	41	<0.05	0.66	<0.2	14	59	6
9/18/78	Not Sampled												
10/16/78	Not Sampled												
11/9/78	Not Sampled												
12/6/78	Not Sampled												

*Major source of total iron is the rust from metal well piping and pumping equipment.

NOTES:
1. Well locations are shown in Figure 2.4-28.
2. Samples collected and analyzed by Commonwealth Edison Company.
3. Units for all constituents except pH in milligrams per liter.
4. An X indicates that the sample was not tested for this constituent.

TABLE 2.4-27 (Cont'd)

WELL 6

DATE	pH	SULPHATE	TOTAL DISSOLVED SOLIDS	TOTAL ALKALINITY	TOTAL HARDNESS	CALCIUM	MAGNESIUM	SOLUBLE IRON	TOTAL* IRON	BORON	CHLORIDE	TOTAL SUSPENDED SOLIDS
12/9/75	7.4	25	442	325	355	72	45	0.75	36.0	<0.2	X	X
1/20/76	7.2	24	388	324	366	72	45	0.34	28.7	<0.2	4	X
2/17/76	7.4	35	398	284	348	75	39	0.09	11.2	<0.2	8	148
3/23/76	7.3	21	472	256	454	85	58	0.05	4.82	0.8	16	16
4/13/76	7.3	32	464	334	406	81	50	0.02	2.96	<0.2	12	28
5/21/76	7.5	38	458	334	400	80	48	<0.02	3.42	<0.2	12	54
6/7/76	7.4	38	458	328	381	80	44	0.02	3.0	0.2	8	36

Sampling of Well 6 discontinued.

*Major source of total iron is the rust from metal well piping and pumping equipment.

NOTES:
1. Well locations are shown in Figure 2.4-28.
2. Samples collected and analyzed by Commonwealth Edison Company.
3. Units for all constituents except pH in milligrams per liter.
4. An X indicates that the sample was not tested for this constituent.

TABLE 2.4-27 (Cont'd)

WELL 7

DATE	pH	SULPHATE	TOTAL DISSOLVED SOLIDS	TOTAL ALKALINITY	TOTAL HARDNESS	CALCIUM	MAGNESIUM	SOLUBLE IRON	TOTAL* IRON	BORON	CHLORIDE	TOTAL SUSPENDED SOLIDS	OIL AND GREASE
7/5/76	7.5	40	501	276	343	75	38	<0.02	<0.02	<0.2	13	6	2
8/11/76	7.4	40	394	274	351	72	41	<0.02	4.70	0.3	9	2	1
9/20/76	7.4	41	416	274	331	61	44	<0.02	0.03	1.9	12	5	1
10/19/76	7.4	34	344	276	351	54	53	<0.02	0.03	0.3	11	2	1
11/8/76	7.3	35	370	264	353	78	39	<0.02	0.09	<0.2	9	2	1
12/6/76	7.4	38	346	258	352	80	37	<0.02	0.05	<0.2	11	2	1
1/18/77	7.4	40	324	260	348	72	40	0.06	0.06	0.2	8	2	1
2/22/77	7.3	41	384	258	356	75	41	<0.05	0.12	<0.2	10	2	1
3/21/77	7.6	63	374	284	354	87	33	<0.05	<0.05	<0.2	12	2	X
4/26/77	7.3	43	360	260	328	70	37	<0.1	<0.1	<0.2	6	2	6
5/10/77	7.5	40	986	252	323	65	39	0.02	0.04	<0.2	7	63	6
6/13/77	7.6	45	336	272	323	62	39	<0.02	<0.02	<0.2	6	2	7
7/19/77	7.3	36	368	240	295	68	31	0.03	2.53	<0.2	3	3	4
8/9/77	7.4	56	320	248	338	70	39	<0.02	0.04	<0.2	4	4	2
9/26/77	7.5	45	380	236	323	80	30	<0.02	<0.02	<0.2	7	2	2

TABLE 2.4-27 (Cont'd)

WELL 7

DATE	pH	SULPHATE	TOTAL DISSOLVED SOLIDS	TOTAL ALKALINITY	TOTAL HARDNESS	CALCIUM	MAGNESIUM	SOLUBLE IRON	TOTAL* IRON	BORON	CHLORIDE	TOTAL SUSPENDED SOLIDS	OIL AND GREASE
10/24/77	7.6	33	352	260	470	84	63	<0.05	<0.05	<0.2	12	2	2
11/28/77	7.3	55	326	252	341	78	35	<0.02	0.05	<0.2	8	2	2
12/19/77	7.4	36	384	256	320	78	30	<0.05	0.23	<0.2	9	2	1
1/30/78	7.5	38	404	266	330	88	27	<0.05	<0.05	<0.2	11	6	1
2/21/78	7.5	37	326	268	342	76	37	<0.05	<0.05	0.4	11	3	4
3/20/78	7.4	36	382	264	368	79	42	<0.05	<0.05	<0.2	11	3	2
4/10/78	7.4	40	356	260	352	74	41	0.05	0.05	0.4	13	2	1
5/22/78	7.4	42	346	270	344	75	38	<0.05	0.06	<0.2	9	<2	2
6/13/78	7.3	42	384	284	372	77	44	<0.05	<0.05	<0.2	15	<2	3
8/1/78	7.5	50	442	280	388	88	41	<0.05	0.14	<0.2	15	2	1
8/29/78	7.5	49	518	256	340	75	37	<0.05	<0.05	<0.2	6	2	1
9/18/78	7.4	50	556	288	402	89	44	<0.05	<0.05	<0.2	17	1	1
10/16/78	7.3	32	462	278	400	85	46	<0.05	0.05	<0.2	X	7	1

TABLE 2.4-27 (Cont'd)

WELL 7

DATE	pH	SULPHATE	TOTAL DISSOLVED SOLIDS	TOTAL ALKALINITY	TOTAL HARDNESS	CALCIUM	MAGNESIUM	SOLUBLE IRON	TOTAL* IRON	BORON	CHLORIDE	TOTAL SUSPENDED SOLIDS	OIL AND GREASE
11/9/78	7.3	51	422	282	396	85	45	<0.05	0.05	<0.2	16	2	1
12/6/78	7.3	45	420	276	386	80	45	<0.05	<0.05	<0.2	17	4	1

*Major source of total iron is the rust from metal well piping and pumping equipment.

NOTES:
1. Well locations are shown in Figure 2.4-28.
2. Samples collected and analyzed by Commonwealth Edison Company.
3. Units for all constituents except pH in milligrams per liter.
4. An X indicates that the sample was not tested for this constituent.

TABLE 2.4-27 (Cont'd)

WELL 8

DATE	pH	SULPHATE	TOTAL DISSOLVED SOLIDS	TOTAL ALKALINITY	TOTAL HARDNESS	CALCIUM	MAGNESIUM	SOLUBLE IRON	TOTAL* IRON	BORON	CHLORIDE	TOTAL SUSPENDED SOLIDS	OIL AND GREASE
7/5/76	7.3	36	454	296	340	72	39	<0.02	1.95	<0.2	8	5	4
8/11/76	7.4	38	426	299	350	72	41	<0.02	0.17	<0.2	3	2	2
9/20/76	7.3	38	382	304	369	98	30	<0.02	5.0	1.0	7	8	<1
10/19/76	7.5	30	328	214	366	68	47	<0.02	4.83	<0.2	<1	4	<1
11/8/76	7.2	36	374	298	367	76	43	<0.02	4.57	<0.2	4	11	<1
12/6/76	7.2	36	342	34	376	78	44	<0.02	5.78	<0.2	6	14	<1
1/18/77	7.0	40	334	300	352	70	43	0.14	9.90	0.2	4	19	<1
2/22/77	7.3	40	392	296	366	77	43	0.08	0.35	<0.2	5	20	<1
3/21/77	7.4	63	364	312	350	72	41	<0.05	5.7	<0.2	5	16	X
4/26/77	7.3	42	376	268	312	70	33	<0.1	0.35	<0.2	3	<2	4
5/10/77	7.7	40	380	292	335	66	41	0.03	6.90	<0.2	7	17	8
6/13/77	7.5	60	382	286	357	74	42	<0.02	0.08	<0.2	3	<2	1
7/19/77	7.4	39	350	298	345	76	62	<0.02	<0.02	<0.2	3	11	2
8/9/77	7.3	50	414	288	362	80	39	<0.02	6.2	<0.2	7	18	3
9/26/77	7.5	35	400	302	355	80	38	0.20	9.9	<0.2	9	16	4

TABLE 2.4-27 (Cont'd)

WELL 8

DATE	pH	SULPHATE	TOTAL DISSOLVED SOLIDS	TOTAL ALKALINITY	TOTAL HARDNESS	CALCIUM	MAGNESIUM	SOLUBLE IRON	TOTAL* IRON	BORON	CHLORIDE	TOTAL SUSPENDED SOLIDS	OIL AND GREASE
10/24/77	7.4	29	370	294	360	80	39	<0.05	6.9	<0.2	6	11	3
11/28/77	7.2	40	356	300	364	74	43	0.09	4.48	<0.2	6	16	<1
12/19/77	7.4	29	388	264	348	74	40	0.18	4.07	<0.2	7	19	1
1/30/78	7.2	32	386	310	356	70	44	<0.05	12.4	<0.2	6	35	1
2/21/78	7.2	32	354	306	360	72	44	<0.05	8.1	<0.2	7	20	11
3/20/78	7.4	36	364	312	384	80	45	0.12	4.95	0.2	2	18	5
4/10/78	2.2	36	348	296	350	71	42	0.05	1.59	0.2	5	3	1
5/22/78	7.5	34	306	310	350	72	41	<0.05	3.89	0.4	7	2	5
6/13/78	7.3	36	350	300	362	73	44	0.16	0.46	<0.2	4	<2	21
8/1/78	7.6	37	370	304	344	70	41	<0.05	1.22	<0.2	2	6	<1
8/29/78	7.4	38	406	304	348	80	36	<0.05	<0.05	<0.2	3	2	<1
9/18/78	7.1	33	426	302	362	75	42	<0.05	2.57	<0.2	3	8	2
10/16/78	7.3	45	378	306	350	72	41	0.06	1.67	<0.2	X	10	5

TABLE 2.4-27 (Cont'd)

WELL 8

DATE	pH	SULPHATE	TOTAL DISSOLVED SOLIDS	TOTAL ALKALINITY	TOTAL HARDNESS	CALCIUM	MAGNESIUM	SOLUBLE IRON	TOTAL* IRON	BORON	CHLORIDE	TOTAL SUSPENDED SOLIDS	OIL AND GREASE
11/9/78	7.2	36	352	308	352	72	42	<0.05	2.06	<0.2	5.5	6	1
12/6/78	7.5	21	352	292	334	64	43	<0.05	2.99	<0.2	3	10	1

*Major source of total iron is the rust from metal well piping and pumping equipment.

NOTES: 1. Well locations are shown in Figure 2.4-28.
2. Samples collected and analyzed by Commonwealth Edison Company.
3. Units for all constituents except pH in milligrams per liter.
4. An X indicates that the sample was not tested for this constituent.

TABLE 2.4-27 (Cont'd)

WELL 9

DATE	pH	SULPHATE	TOTAL DISSOLVED SOLIDS	TOTAL ALKALINITY	TOTAL HARDNESS	CALCIUM	MAGNESIUM	SOLUBLE IRON	TOTAL* IRON	BORON	CHLORIDE	TOTAL SUSPENDED SOLIDS	OIL AND GREASE
7/5/76	7.5	43	444	272	330	69	38	<0.02	0.04	0.5	12	4	5
8/11/76	7.5	41	438	262	334	72	38	<0.02	<0.02	0.2	6	3	1
9/20/76	7.6	41	384	262	319	62	40	<0.02	<0.02	0.7	10	1	1
10/19/76	7.6	34	368	366	335	68	40	<0.02	<0.02	0.8	8	2	1
11/8/76	7.3	40	396	256	343	75	38	<0.02	<0.02	0.7	7	3	1
12/6/76	7.4	40	356	250	331	72	37	<0.02	<0.24	<0.2	9	2	1
1/18/77	7.2	40	372	256	328	69	38	<0.04	0.07	0.2	6	2	1
2/22/77	7.4	42	428	250	333	71	38	<0.05	6.25	0.5	9	3	1
3/21/77	7.6	58	352	250	310	76	29	<0.05	<0.05	<0.2	6	2	X
4/26/77	7.5	43	398	252	332	67	40	<0.01	<0.1	<0.2	6	2	4
5/10/77	7.7	55	392	248	315	68	35	0.03	0.32	<0.2	7	2	7
6/13/77	7.5	54	360	252	323	69	37	<0.02	0.08	<0.2	6	2	6
7/19/77	7.4	45	476	250	327	67	39	<0.02	<0.02	<0.2	5	2	1
8/9/77	7.6	40	404	260	350	75	39	<0.02	0.09	<0.2	9	2	11
9/26/77	7.6	35	368	244	347	74	39	0.04	0.04	<0.2	6	2	2

TABLE 2.4-27 (Cont'd)

WELL 9

DATE	pH	SULPHATE	TOTAL DISSOLVED SOLIDS	TOTAL ALKALINITY	TOTAL HARDNESS	CALCIUM	MAGNESIUM	SOLUBLE IRON	TOTAL* IRON	BORON	CHLORIDE	TOTAL SUSPENDED SOLIDS	OIL AND GREASE
10/24/77	7.5	30	394	260	334	75	35	<0.05	<0.05	<0.2	8	2	5
11/28/77	7.4	40	356	252	338	76	36	<0.02	<0.05	0.5	5	3	1
12/19/77	7.5	36	374	236	316	67	36	<0.05	<0.05	<0.2	6	4	1
1/30/78	7.4	36	376	264	328	70	37	<0.05	<0.05	<0.2	5	2	1
2/21/78	7.5	29	330	256	340	69	41	<0.05	0.08	0.2	10	4	4

*Major source of total iron is the rust from metal well piping and pumping equipment.

NOTES:
1. Well locations are shown in Figure 2.4-28.
2. Samples collected and analyzed by Commonwealth Edison Company.
3. Units for all constituents except pH in milligrams per liter.
4. An X indicates that the sample was not tested for this constituent.

TABLE 2.4-28

SITE AREA GROUNDWATER LEVELS

WATER QUALITY MONITORING PROGRAM

GALENA-PLATTEVILLE DOLOMITES

	WELL 1		WELL 2		WELL 3		WELL 5		WELL 6	
DATE	APPROX. DEPTH	APPROX. ELEV.	APPROX. DEPTH	APPROX. ELEV.	APPROX. DEPTH	APPROX. ELEV.	APPROX. DEPTH	APPROX. ELEV.	APPROX. DEPTH	APPROX. ELEV.
12/9/75	45	741	43	746			81	769	79	754
1/20/76	60	726			40	853			90	743
2/17/76	50	736	55	734			100	750	90	743
3/23/76	51	735	40	749	55	838	95	755	70	763
4/13/76	28	758	48	741	38	855	85	765	63	770
5/21/76	28	758	48	741	36	857	78	772	61	772
6/7/76	33	753	48	741	38	855	83	767	68	765
7/5/76	38	748			54	839	96	754		
8/11/76	41	745			56	837	97	753		
9/20/76	40	746			55	838	95	755		
10/19/76	41	745			57	836				
11/8/76	40	746			48	845	96	754		
12/6/76	39	747			57	836	97	753		

TABLE 2.4-28 (Cont'd)

DATE	WELL 1 APPROX. DEPTH	WELL 1 APPROX. ELEV.	WELL 2 APPROX. DEPTH	WELL 2 APPROX. ELEV.	WELL 3 APPROX. DEPTH	WELL 3 APPROX. ELEV.	WELL 5 APPROX. DEPTH	WELL 5 APPROX. ELEV.	WELL 6 APPROX. DEPTH	WELL 6 APPROX. ELEV.
1/18/77					56	837				
2/22/77	39	747			51	842	91	759		
3/21/77	40	746			50	843	91	759		
4/26/77	42	744			52	841	100	750		
5/10/77	45	741			55	838	95	755		
6/13/77	47	739			59	834	103	747		
7/19/77					58	835	103	747		
8/9/77	42	744			50	843	95	755		
9/26/77					59	834	99	751		
10/24/77	39	747			59	834	100	750		
11/28/77	33	753			54	839	92	758		
12/19/77	32	754			53	840	95	755		
1/30/78	30	756			53	840				
2/21/78	36	750			54	839				
3/20/78	27	759			48	845				

TABLE 2.4-28 (Cont'd)

DATE	WELL 1 APPROX. DEPTH	WELL 1 APPROX. ELEV.	WELL 2 APPROX. DEPTH	WELL 2 APPROX. ELEV.	WELL 3 APPROX. DEPTH	WELL 3 APPROX. ELEV.	WELL 5 APPROX. DEPTH	WELL 5 APPROX. ELEV.	WELL 6 APPROX. DEPTH	WELL 6 APPROX. ELEV.
4/10/78	26	760					89	761		
5/22/78	29	757			51	842	88	762		
6/13/78	27	759			49	844	98	752		
8/1/78					49	844	99	751		
8/29/78	37	749			53	840				
9/18/78					37	846				
10/16/78	32	754			53	840				
11/9/78	30	756			51	842				
12/6/78	36	750			53	840				

NOTES:
1. Depths are the measured depths to the groundwater in feet.
2. Elevations are in feet MSL.
3. Well locations are shown on Figure 2.4-28.
4. Water quality data on samples from these wells is presented in Table 2.4-27.
5. Missing data indicates groundwater levels were not measured in that well on that date.
6. Groundwater level measurements were discontinued in Wells 2 and 6 in June 1976.

TABLE 2.4-29

DEEP WELL CONSTRUCTION DETAILS

WELL [1]	LOCATION	ELEVATION ft, MSL	DATE COMPLETED	WELL DEPTH ft	DIAMETER inches	PUMP TEST DATA DATE	SPECIFIC CAPACITY, gpm/ft	PUMPING RATE, gpm	DURATION, [3] HOURS	STATIC WATER LEVEL ft	(date)	COMMENTS
Deep Well No. 1 (east well)	T24N, R10E, 215'S, 1010' W, NE NE NE NE Sec 24	875.5[2] Original Grade	December 1974	0-20' 0-241' 241-462' 462-834'	24" Cased 20" Cased 19" Open 15" Open	12/74 11/78	10.3 8.5	620 840	12 3 1/2	185 186	(12/74) (11/78)	Principal producing units are St. Peter Sandstone, and upper portion of Ironton-Galesville Sandstones.
	31 + 03 N 45 + 20 E (Plant Coordinates)	876	June 1980	0-700' 834-1500'	16" Cased 15" Open	06/80 07/80	12.3 13.2	1300 790	12 24	217 224	(06/80) (07/80)	Well modified in 1980.[4] Principal producing units are Ironton-Galesville Sandstones and Mt. Simon Aquifer.

NOTES
1. Well No. 2 was drilled before well No. 1; therefore, early drilling records on file with the Illinois State Geological Survey have reversed well numbers, i.e., well No. 2 is "well No. 1."
2. Measured at top of pitless adapter casing.
3. Duration refers to portion of test used to calculate specific capacity. Total test duration may have been longer than indicated.
4. Pumping equipment consists of 10-stage Byron Jackson submersible pumps installed 412 ft deep. Maximum pumping level is 350 ft. Available drawdown is approximately 125 ft.

TABLE 2.4-29, (Cont'd)

WELL [1]	LOCATION	ELEVATION ft, MSL	DATE COMPLETED	WELL DEPTH ft	DIAMETER inches	PUMP TEST DATA DATE	SPECIFIC CAPACITY, gpm/ft	PUMPING RATE, gpm	DURATION, [3] HOURS	STATIC WATER LEVEL ft	(date)	COMMENTS
Deep Well No. 2 (west well)	T24N, R10E 365'S, 520'E, NW NW NW NE Sec 24	870.4[2] Original Grade	October 1974	0-20' 0-230' 230-549' 549-853'	24" Cased 20" Cased 19" Open 15" Open	11/74	9.6	1150	24	187 181	(11/74) (11/78)	Principal producing units are St. Peter Sandstone, and upper portion of Ironton-Galesville Sandstones
	29 + 16 N 33 + 90 E (Plant Coordinates)	870	December 1979	0-700' 853-1500'	16" Cased 12" Open	12/79	12.2	1210	9 1/2	206 218	(12/79) (07/80)	Well modified in 1979.[4] Principal producing units are Ironton-Galesville Sandstones and Mt. Simon Aquifer.

NOTES

1. Well No. 2 was drilled before well No. 1; therefore, early drilling records on file with the Illinois State Geological Survey have reversed well numbers, i.e., well No. 2 is "well No. 1."
2. Measured at top of pitless adapter casing.
3. Duration refers to portion of test used to calculate specific capacity. Total test duration may have been longer than indicated.
4. Pumping equipment consists of 10-stage Byron Jackson submersible pumps installed 412 ft deep. Maximum pumping level is 350 ft. Available drawdown is approximately 125 ft.

TABLE 2.4-30

PARAMETERS USED IN STRUCTURAL ANALYSIS

OF RIVER SCREEN HOUSE

PARAMETERS	COMBINED EVENT FLOOD*	FLOOD OF RECORD**	BREAKING WAVE
Water Level (feet)	698.7	682.0	-
Average Depth (feet)	20.5	10.5	7.1
Fetch Length (miles)	1.05	1.05	1.05
Overland Wind Speed (mph)	40	60	53
Significant Wave Height (feet)	2.05	3.2	2.75
Maximum Wave Height (feet)	3.42	5.3	4.6
Wave Period (seconds)	2.65	3.2	3.0
Breaking (B) or Nonbreaking (NB)	NB	NB	B
Hydrostatic Force (lb/ft^2)	44820 (max.)	14690 (max.)	3060 (max.)
Line of Action Above Grade (feet)	12.6	7.2	3.3
Hydrodynamic Force (lb/ft^2)	8065 (max.)	5560 (max.)	23600 (max.)
Line of Action Above Grade (feet)	21.5	13.9	7.1

*See Subsection 2.4.3.7.

**Flood of Record is used in lieu of 25-year flood.

TABLE 2.4-31

SUSPENDED SEDIMENT CONCENTRATIONS

DATE	SECTION	LOCATION	DISTANCE FROM LEFT BANK (ft)	CONCENTRATION (ppm)	DISCHARGE (cfs)
7-29-80	3	Centerline of Intake	173	73.4	3219
			185	65.8	
			200	72.6	
7-31-80	2	464 ft upstream of Section 1	155	55.3	2800
			181	56.7	
7-31-80	4	457 ft downstream of Section 1	88	166.3	2500
			107	79.8	
			150	40.9	
10-7-80	1	947 ft upstream of Section 3	131	76.5	8245
			157	71.4	
			186	67.2	

TABLE 2.4-32

MONTHLY FLOWS AT INTAKE

MONTH	LOWEST MEAN MONTHLY FLOW (cfs)	AVERAGE MEAN MONTHLY FLOW (cfs)
January	977	4100
February	1263	5704
March	1855	9227
April	2660	8170
May	1390	5399
June	1162	4648
July	826	3557
August	812	2851
September	932	2604
October	1129	3193
November	1314	3525
December	1317	3475

2.5 GEOLOGY, SEISMOLOGY, AND GEOTECHNICAL ENGINEERING

2.5.1 Basic Geologic and Seismic Information

The Byron Station, Units 1 and 2, is located in north central Illinois about 2 miles east of the Rock River and approximately 3 miles southwest of Byron in Ogle County. The site is located within Rockvale Township (T.24N., R.10E.) and includes Section 13 and portions of Sections 12 and 24. The regional site map is shown on Figure 2.5-1.

The site is divided into a flat upland portion containing the plant buildings and a small portion 2 miles to the west, in the Rock River valley, containing the intake works. The maximum topographic relief between the two parts is about 225 feet.

The upland portion of the Byron site is developed on flat-lying Ordovician dolomites covered by 4 to 37 feet of soil overburden, predominantly Pleistocene-age alluvium, loess, till, and residuum. Soil deposits along the Rock River, in the area of the river intake works, consist of approximately 115 feet of Holocene and Pleistocene age alluvial material overlying the St. Peter Sandstone of Ordovician age.

The site is located on the northern flank of the Illinois Basin near the crest of the Wisconsin Arch. The bedrock units at the Byron site belong to the Ordovician-age Galena Group dolomites. These unit dip gently to the south-southeast towards the Illinois Basin, at angles of less than 1°.

As predicted in the PSAR, the Ordovician age Dunleith Formation provided an excellent foundation base, after remedial treatment (grouting is discussed in FSAR Attachments 2.5A and 2.5B). The only significant new situation that developed in construction was the discovery of small displacement faults during the main plant excavation. Analysis of these small displacement faults by geologists from the Illinois State Geological Survey, Dames & Moore, and Sargent & Lundy have indicated that these features are noncapable (see FSAR Attachment 2.5C).

2.5.1.1 Regional Geology

2.5.1.1.1 Geologic History

2.5.1.1.1.1 General

The study of geologic history provides an insight as to the tectonic stability of a region and a better understanding of stratigraphic relationships between various soil and rock units. It also furnishes correlative data which assists in the interpretation of events in adjacent regions.

An accurate interpretation of geologic history is the result of years of accumulative effort. It is based on numerous examinations of soil and rock units in exposures and from borings with regard to lithology and fossil content. Comparisons are drawn with events observed in present-day environments.

A generalized regional stratigraphic column for north-central Illinois is presented on Figure 2.5-8. It is composite in nature. The entire series of stratigraphic units may not be encountered at any given locality. Individual units are discussed in the following paragraphs. The ages for the geologic periods discussed below are taken from Faul (Reference 1).

2.5.1.1.1.2 Precambrian (Greater than Approximately 600 Million Years B.P.)

The Precambrian is the oldest recognized division of geologic time and its history is at best obscure. In northern Illinois, Precambrian rocks lie some 2000 to 5000 feet below sea level. Their nature and history must of necessity be based on boring samples and observations of exposures in more distant regions. Twenty-eight drill holes in Illinois have reached Precambrian basement. The holes most commonly encountered medium to coarse-grained granite. Other rock types reported are quartz monzonite, rhyolite porphyry, and felsite (Reference 2).

In central and northern Wisconsin, Precambrian rocks are exposed at the surface. Here they record alternating periods of sedimentary deposition and erosion, mountain building, metamorphism, and igneous activity. This was followed by a long period of subaerial erosion which reduced the cores of the Precambrian mountains and formed a gently sloping peneplain of regional extent. It is inferred that similar events took place throughout northern Illinois.

2.5.1.1.1.3 Cambrian (Approximately 500 to Approximately 600 Million Years B.P.)

Due to the absence of early and middle Cambrian rocks both in northern Illinois and in Wisconsin, it is assumed that the long periods of erosion which took place in late Precambrian time continued through early and middle Cambrian time. By late Cambrian time, a crustal flexure began to form the Central Interior Basin. The lowlands were submerged by shallow seas which advanced from the south. Several thousand feet of sediments, comprising the Croixian Series, were deposited in northern Illinois. Conditions changed from time to time producing a wide variation in lithologies. Cambrian time ended with an uplift of the region above sea level followed by a period of erosion.

The area of central Wisconsin was probably uplifted several times during the Paleozoic Era. Initial movements may have taken place during Cambrian time to form the Wisconsin Dome, but evidence of time and spatial relations is scarce.

2.5.1.1.1.4 Ordovician (430 ± 10 to Approximately 500 Million Years B.P.)

The Ordovician period began with a readvance of the sea. Initial deposition consisted primarily of calcareous materials followed by alternating periods of clastic and calcareous deposition, all of which comprise the Canadian Series.

At the end of Canadian time, regional uplifting occurred and wide-spread erosion was initiated. A well defined river system was developed in portions of northern Illinois. Deep erosional valleys were produced which possibly cut into the underlying Cambrian sediments. During this period, significant uplift of the Wisconsin Dome is also recognized.

Following the long period of erosion, Champlainian time was initiated by an advance of the sea over existing erosional topography. Unconsolidated materials were reworked and sand was deposited under conditions which remained stable over a vast area. The period of sand deposition was followed by relatively brief emergence. Again the sea advanced to initiate some reworking. A long period of calcareous deposition in clear seas with subsequent emergence completed the Champlainian Series.

Emergence was apparently brief and the region was again submerged by a sea which advanced from the north. Deposits of the Cincinnatian Series initially consisted of thick accumulations of silt and mud, probably in shallow, turbid seas. Later, the seas cleared and calcareous deposits were formed. Finally, turbid conditions returned as evidenced by the deposition of more silt and mud.

The Kankakee Arch may have begun to form during the Ordovician. Structural relief is believed to have occurred by subsidence of the Illinois and Michigan Basins while the Kankakee Arch remained more or less stable. This stability has been theoretically attributed to the presence of underlying eroded cores of Precambrian mountain ranges.

Ordovician time ended by the emergence of the region above sea level followed by a long period of erosion.

2.5.1.1.1.5 Silurian (400 ± 10 to 430 ± 10 Million Years B.P.)

After an erosional interval of long duration, northern Illinois was again inundated during Alexandrian time by marine waters that advanced from the Gulf of Mexico and eventually connected with seas to the north. Deposition began with clastic sediments in shallow seas which became very clear before the end of Alexandrian time, as indicated by very pure carbonates.

Deposition during the following Niagaran time is represented by a thick carbonate sequence characterized by reefs. This period of deposition was followed by an uplift and erosion which began late in the Silurian and continued into Devonian time.

2.5.1.1.1.6 Devonian (340 ± 10 to 400 ± 10 Million Years B.P.)

Following the deposition of the Silurian beds, a second uplifting of the Wisconsin Dome is recognized. This time, an arch was formed which extended southeastward from Wisconsin into Illinois, almost to the city of Kankakee. This structure is called the Wisconsin Arch.

Erosion in northern Illinois continued from Silurian through early Devonian time. The middle Devonian deposition began with a major transgression of the sea. The Kankakee Arch acted as a barrier for a time. Sedimentation began with an accumulation of calcareous materials and ended with relatively thick accumulations of silt and mud in late Devonian time.

The Devonian period ended with the emergence of the region above sea level and subsequent erosion which appears to have removed much of the Devonian rocks in northern Illinois.

2.5.1.1.1.7 Mississippian (320 ± 10 to 340 ± 10 Million Years B.P.),

The events which took place in northern Illinois between close of the Devonian and the beginning of the Pennsylvanian period are not clearly known.

Widespread Mississippian age deposits are not found in extreme northern Illinois. It is postulated that Mississippian seas never advanced completely over the region. Where observed, the deposits consist predominantly of marine carbonates. Initial deposits, however, were silt and mud. A somewhat gradual transition is indicated in adjacent regions between early Mississippian and late Devonian deposition.

During this span of time, the major folding of the LaSalle Anticline took place, probably during late Mississippian, along with additional movements of the Kankakee Arch. The Sandwich Fault was probably formed as a result of subsequent relaxational movements. These tectonic movements were accompanied by wide-spread erosion which removed existing Mississippian and Devonian-aged deposits, throughout most of northern Illinois. To the north, the Wisconsin Dome had become prominent by the close of the Mississippian time.

2.5.1.1.1.8 Pennsylvanian (270 ± 5 to 320 ± 10 Million Years B.P.)

In Pennsylvanian time, conditions controlling sedimentation were considerably different from those in which earlier Paleozoic sediments were deposited. Throughout Pennsylvanian time, highland areas existed along eastern and southern parts of North

America. The interior of the continent was a plain that was repeatedly submerged by the sea or lay a short distance above it. When the sea submerged the plain, streams from the highland areas carried rock debris into it. The sediments accumulated in a marine environment. As the sea receded, deposition continued but the deposits accumulated in paludal environments. The newly emerged plain became covered by swamps which extended unbroken for hundreds of miles. Vegetation flourished and accumulated in thick deposits to form the coal layers which exist today. Eventually the sea returned to initiate another cycle of sedimentation. Each cycle is therefore partly marine and partly paludal in origin. Numerous similar cycles of deposition, many of which are separated by localized erosional unconformities, have been recorded from the Pennsylvanian stratigraphy.

Pennsylvanian deposits thin over the LaSalle Anticline indicating some continued tectonic movements. Late or post-Pennsylvanian time marked the climax of the LaSalle Anticlinal folding. Probably all the structural units involved in the late Mississippian period of folding were again affected.

2.5.1.1.1.9 Permian (225 ± 5 to 270 ± 5 Million Years B.P.)

No deposits of Permian age have been found in the regional area. The apparent absence of deposits indicates that the Permian was a period of nondeposition or that Permian deposits in the area were subsequently eroded.

2.5.1.1.1.10 Triassic (190 ± 5 to 225 ± 5 Million Years B.P.)

There are no deposits of Triassic age in the regional area. This was largely a period of erosion (Reference 3).

2.5.1.1.1.11 Jurassic (135 ± 5 to 190 ± 5 Million Years B.P.)

There are no deposits of Jurassic age in the regional area. This was largely a period of erosion (Reference 3).

2.5.1.1.1.12 Cretaceous (65 ± 2 to 135 ± 5 Million Years B.P.)

Cretaceous deposits are not encountered in northern Illinois. It seems likely that Cretaceous seas did not advance much beyond western central Illinois where relatively small areas of Cretaceous rocks are known to be present and unconformably overlie Pennsylvanian strata.

Geologic evidence suggests that northern Illinois existed as a low, stable land mass for over 200,000,000 years (since the start of the Mesozoic era), while the Appalachian Mountains, Rocky Mountains, and other structural features in North America were being formed or undergoing additional movements.

2.5.1.1.1.13 Quaternary (Present to 2 ± 1 Million Years B.P.)

Glaciation in northern Illinois began with the Pleistocene some 1,000,000 years ago. Four major glacial advances invaded the region according to Willman and Frye (Reference 4).

The ice fronts probably advanced and retreated several times during each of these glacial periods, but the record is scant for the Kansan and absent for the Nebraskan, the oldest advance. Deposits of the last two major advances, the Illinoian and the Wisconsinan, are well documented in northern Illinois (Reference 4). The areas covered by ice during each of the advances, even from the same sources, were not identical. The result is a complex series of partially overlapping deposits and associated features, each of which was modified to a greater or lesser degree by subsequent events.

During each advance, the glaciers eroded some of the preexisting deposits. Debris was also deposited from the ice in the form of till plains, moraines, and outwash during the advance and retreat of the ice. Melt water flowing away from the glacier front was also responsible for eroding, reworking, and redepositing many of these materials. Windblown silt (loess), derived from the outwash sediments in the valleys of the melt water streams, was widely distributed over the land surface well beyond the glacier front. Sand dunes were also developed locally.

Between glaciations, the climate returned to more temperate conditions. Streams developed their drainage systems; at least initially, their positions were largely controlled by the character of the surface left by the retreating glaciers. As these materials were exposed, weathering processes began modifying them. The thickness and character of the resulting soils are largely a function of climate, topographic position, vegetation, and duration of the interglacial period.

Deposits of the Wisconsinan glacial advance are relatively well preserved since it was the last of the great ice sheets to invade northern Illinois. Many of the present-day land forms are attributable to this last advance.

2.5.1.1.2 Physiography

The northern portion of the midwestern United States is located in the Central Lowlands Physiographic Province. This physiographic province has been divided into several physiographic sections. Parts of northern Illinois are located in the Wisconsin Driftless Section, the Till Plains Sections, and the Great Lake Section (Figure 2.5-9).

The site is located within the Till Plains Section. The Till Plains Section is characterized in general by the presence of glacial deposits overlying the bedrock surface. Local outcrops of bedrock are present. The Till Plains Section in Illinois is further subdivided into the following physiographic subsections: the Rock River Hill Country, the Green River Lowland, the Bloomington Ridged Plain, the Galesburg Plain, the Kankakee Plain, and the Springfield Plain.

The site area is located in the Rock River Hill Country physiographic subsection. The Rock River Hill Country is characterized by gently rolling, dissected uplands covered by thin deposits of glacial drift overlain by a thin cap of loess. The southwest-trending Rock River valley passes through the eastern portion of the subsection. Bedrock is exposed locally along the Rock River and along small tributary streams and valleys of the Rock River.

2.5.1.1.3 Stratigraphy

2.5.1.1.3.1 Soil Units

The soil units in the region adjacent to the plant site area are generally relatively thin and locally absent. They include alluvial deposits associated with the rivers and streams in the area, glacial deposits of till and outwash generally located in the upland areas, thin loess deposits which overlie the till, and locally some thin residual soils developed from the weathering of the bedrock.

2.5.1.1.3.2 Rock Units

The distribution of the rock units which form the bedrock surface within the region is shown on Figure 2.5-10. The rock units include a sedimentary sequence of Cretaceous-, Pennsylvanian-, Mississippian-, Devonian-, Silurian-, Ordovician-, and Cambrian-aged strata and an igneous and metamorphic complex of Precambrian-aged rocks. A regional geologic section, south of the site (Figure 2.5-11), shows the relationship between the sedimentary rock strata.

The sedimentary sequence in northern Illinois in the proximity of the site includes Ordovician- and Cambrian-aged strata. These strata consist of 2000 to 3000 feet of dolomites, sandstones, and shales. The Precambrian basement in northern Illinois consists of granites and granodiorites (Reference 2, p. 4). The relationship of these rock units is shown on the regional stratigraphic column (Figure 2.5-8).

2.5.1.1.4 Structure

The site of the Byron Station lies within the Central Stable Region tectonic province of the North American continent. This tectonic region is characterized by a sequence of southward-thickening sedimentary strata overlying the Precambrian basement and was subjected to a series of vertical crustal movements forming broad basins and arches during Paleozoic and early Mesozoic time. The arches and basins have been modified by local folding and faulting. Major geologic structures are shown on Figures 2.5-12 and 2.5-13. The relationship of these structures to regional seismicity is discussed in Subsection 2.5.2.3.

2.5.1.1.4.1 Folding

The distribution of major folds in the region is shown on Figures 2.5-12, 2.5-14, and 2.5-15 and their characteristics are presented in Table 2.5-1. The knowledge of these structural features is based on surface and/or subsurface geological data. The geologic age of the most recent movement associated with these major structural features is considered to be pre-Cretaceous with the major movement occurring in Paleozoic time. The direction and amount of regional dip of the strata in northern Illinois vary. In the vicinity of the site area, the strata dip southward toward the Illinois Basin at 1° or less.

Kankakee Arch

The Kankakee Arch is located southeast of the site area and trends in a northwesterly direction (Figure 2.5-12).

Wisconsin Arch

The Wisconsin Arch is a south-to southeast-trending extension of the Wisconsin Dome. It can be traced into Illinois to the vicinity of the city of Kankakee where it appears to connect with the Kankakee Arch of Illinois and Indiana. The Wisconsin Arch has a Precambrian core and is believed to be the result of crustal uplift, whereas the Kankakee Arch acquired its structural relief chiefly by greater subsidence of the structural basins which lie on either side of it.

LaSalle Anticlinal Belt

The LaSalle Anticlinal Belt is an en echelon belt of anticlines trending northwest which extends over 200 miles from southeastern Illinois to northern central Illinois. The Ashton Arch and the Oregon Anticline (Figure 2.5-14) may be extensions of the LaSalle Anticlinal Belt. The major flexure of the LaSalle Anticlinal Belt occurred during Pennsylvanian time.

Illinois Basin

The Illinois Basin is oval shaped. It as a major axis, trending approximately N25° W, which is approximately 350 miles long, and a minor axis which is approximately 250 miles long. The deepest part of the basin is in southeastern Illinois.

To the north, the Illinois Basin rises gently to the Wisconsin Arch. To the northeast, the Illinois Basin is separated from the Michigan Basin by the Kankakee Arch. To the east, the Illinois Basin rises gently to the Cincinnati Arch (which is outside of the regional area). To the south, the Illinois Basin rises gently to the Pascola Arch (which is outside of the regional area). To the southwest the Illinois Basin is bordered by the Ozark uplift (which is also outside of the regional area).

The Illinois Basin began to form in the Cambrian and continued to develop intermittently until the end of the Pennsylvanian (Swann and Bell, Reference 5). The depositional center of the basin migrated throughout this time span.

Ashton Arch

Figures 2.5-14 and 2.5-15 show the detail of some of the structural features in northern Illinois. Figure 2.5-14 shows the structure contours on the top of the Galena Group. From this figure it is seen that the western end of the Ashton Arch and the western flank of the Oregon Anticline dip into the Polo Basin. The Ashton Arch is a broad anticline located on the southern side of the Sandwich Fault at the northern end of the LaSalle Anticline. McGinnis (Reference 6) interpreted the Ashton Arch as being a horst (uplifted fault block) and referred to it as the Ottawa Horst.

Plum River Fault Zone

Figure 2.5-15 shows the structural features along the Plum River Fault Zone in Illinois as indicated by the structure contours on the top of the Glenwood Formation. Four minor structural features are located successively from west to east along the fault zone, the Uptons Cave Syncline, the Forreston and Brookville Domes, and the Leaf River Anticline. The Forreston and Brookville Domes were previously considered to be a single domal structure called the Brookville Dome until subsequent drilling indicated the presence of two domal structures. All four of these minor structures are considered to be associated with the development of the Plum River Fault Zone (Kolata and Buschbach, Reference 7). The Plum River Fault Zone formed in the interval from post-Silurian to pre-Pleistocene time. Based on regional geologic history, the Plum River Fault Zone probably developed at the same time that major movements were occurring on other structures in the region, which was near the beginning of Pennsylvanian time and again in post-Pennsylvanian time (Kolata and Buschbach, Reference 7).

Herscher Dome

The Herscher Dome is located approximately 100 miles southeast of the site (Figure 2.5-14). It is an asymmetrical anticlinal structure about 3 miles wide east-west and 5 miles long north-south with over 250 feet of closure. As in other echelon structures in the LaSalle Anticlinal Belt, the strata on the Herscher Dome dip steeply on the southwest side, but more gently on the northeast side (Reference 8).

Downs Anticline

To the south-southeast of the site about 90 miles is a small flexure trending parallel to the LaSalle Anticlinal Belt known as the Downs Anticline (Figure 2.5-12).

Mattoon Anticline

The Mattoon Anticline trends roughly north-south, and is located approximately 160 miles southeast of the site (Figure 2.5-12).

Louden Anticline

The Louden Anticline is located approximately 200 miles south of the site (Figure 2.5-12). It trends north-south and extends from the northern county line of Marion County through east-central Fayette County, Illinois. It is approximately 19 miles long.

Salem Anticline

The Salem Anticline trends approximately parallel to the Louden Anticline, and extends from central Jefferson County, Illinois, to central Marion County, Illinois (Figure 2.5-12). It is approximately 25 miles in length.

Tuscola Anticline

The Tuscola Anticline is one of the many subsidiary structures of the LaSalle Anticlinal Belt (Clegg, Reference 9, Figure 2.5-12). It extends south-southeastward from north of Tuscola in Douglas County to near Charleston in Coles County, Illinois. The anticline plunges southeastward and is broader at the north than at the south (Clegg, Reference 9).

Murdock Syncline

The Murdock Syncline is east of the Tuscola Anticline and shares a common flank with it (Clegg, Reference 9, Figure 2.5-12). The exact extent of the structure is unknown. It probably dies out to the north in Champaign County, Illinois, approximately 160 miles from the site (Clegg, Reference 9). To the south it can be traced only to the vicinity of Charleston, Coles County, Illinois (Clegg, Reference 9).

Marshall Syncline

The Marshall Syncline trends approximately north-south and is located approximately 160 miles from the site (Figure 2.5-12). It is an asymmetrical fold with a comparatively steep west flank (Clegg, Reference 9).

Mississippi River Arch

The Mississippi River Arch is a broad arch trending roughly parallel to the Mississippi River approximately 100 miles southwest of the site.

Pittsfield and Lincoln Anticlines

Located approximately 180 miles to 200 miles southwest of the site and near the Mississippi River are the Pittsfield and Lincoln Anticlines. These two folds parallel each other and trend northwest-southeast.

Mineral Point and Meekers Grove Anticline

The Mineral Point Anticline and Meekers Grove Anticline are located in southwest Wisconsin, approximately 50 to 70 miles northwest of the site and trend roughly east-west.

Baraboo, Fond du Lac, and Waterloo Syncline

Also located in southern Wisconsin are three synclinal structures, the Baraboo Syncline, Fond du Lac Syncline, and Waterloo Syncline. These synclines trend east-west to northeast-southwest and are located about 80 to 110 miles north of the site.

2.5.1.1.4.2 Faulting

2.5.1.1.4.2.1 General Statement

The distribution of major faults in the region is shown on Figure 2.5-13 and their characteristics are presented in Table 2.5-2. The Sandwich Fault and the Plum River Fault Zone are the two major faults in the proximity of the site area.

2.5.1.1.4.2.2 Sandwich Fault Zone and Plum River Fault Zone

The Sandwich Fault Zone trends northwest through northern Illinois. It is mapped on the surface and in the subsurface for a distance of approximately 85 miles. It is an essentially vertical fault with a maximum displacement of approximately 900 feet (Reference 10). The northeastern side has moved down relative to the southwestern side. Movements along the fault zone occurred in the interval between post-Silurian and pre-Pleistocene time. No rocks of intervening ages are present, which prevents better definition of the movements. However, major movements along the fault zone may have been contemporaneous with folding of the LaSalle Anticlinal Belt (Reference 11) during the late Paleozoic.

The Plum River Fault Zone (formerly the Savanna Fault and Savanna Anticline) is generally east-west trending zone of high angle, possibly en echelon faults extending from Leaf River (Ogle Co.), Illinois, to southwest of Maquoketa (Jackson Co.), Iowa

(Reference 7). The fault zone is less than 0.5 mile wide. Vertical displacement along the fault is 100 to 400 feet, north side down. The vertical displacement on the eastern part of the fault, 5.3 miles from the Byron Station, is approximately 100 feet (Reference 7, and Figure 2.5-15). The age of movement has been limited to post-middle Silurian to pre-middle Illinoian (Reference 7). Four minor structural features are associated with the fault zone: the Forreston Dome, the Brookville Dome, the Leaf River Anticline, and the Uptons Cave Syncline. The age of all faults in northern Illinois as determined by the Illinois Geological Survey and Dames & Moore is presented in Table 2.5-2.

2.5.1.1.4.2.3 <u>Oglesby and Tuscola Faults</u>

The Oglesby Fault and the Tuscola Fault were postulated by Green (Reference 12). According to Green, the Oglesby Fault and the Tuscola Fault occur on the western flank of the LaSalle Anticline. Studies by the Illinois State Geological Survey have indicated that the areas where the faults are postulated have dips steeper than the regional dip. No evidence has been found confirming major faulting along the trends of the postulated Oglesby and Tuscola Faults (Reference 13).

2.5.1.1.4.2.4 <u>Chicago Area Faults</u>

2.5.1.1.4.2.4.1 <u>Chicago Area Basement Fault Zone</u>

On the basis of gravity and seismic geophysical evidence, McGinnis (Reference 14) postulated a basement fault zone in the metropolitan Chicago area north of and about parallel to the Sandwich Fault Zone. The presence of the fault has not been verified.

2.5.1.1.4.2.4.2 <u>Chicago Area Minor Faults</u>

As a result of a recent seismic survey in the metropolitan Chicago area, 25 faults were inferred with displacements up to 50 feet (Reference 15). None of these involves wide shear zones or detectable scarps on the rock surface. Faults that have been observed in natural outcrops and quarries in the Chicago area have displacements from a few inches to a few feet, but most are less than 1 foot (Reference 15). These faults are not the same as those inferred from the geophysical survey.

2.5.1.1.4.2.5 <u>Postulated Wisconsin Faults</u>

Thwaites' map of the buried Precambrian surface in Wisconsin (Reference 16) postulates the existence of several faulted areas in the southern and eastern sections of the state. Ostrom (Reference 17) stated that Thwaites' map is diagrammatic and does not represent a detailed study of each fault. Four of the postulated faults, named by Dames & Moore as the Janesville, Appleton, Waukesha, and Madison Faults, have a noticeable difference in the elevation of the basement, which was interpreted by

Thwaites to be the result of faulting. It is now believed (Ostrom, Reference 17) that this difference in elevation of the basement is not due to faulting, but due to topographic relief on an erosional basement surface. Another fault depicted by Thwaites is located approximately 200 miles north-northeast of the site. This postulated fault was named the Green Bay Fault by Dames & Moore, and trends northeast-southwest. Ostrom (Reference 17) believes that this feature is not due to faulting, but is a reef structure in the Silurian-age strata.

2.5.1.1.4.2.6 Mifflin Fault and Royal Center Fault

The Mifflin Fault is located in Iowa and La Fayette Counties, Wisconsin, approximately 75 miles northwest of the site. The fault trace is approximately 10 miles long, and strikes N40° W (Reference 18). The southwest side of the fault is downdropped at least 65 feet, and there is about 1000 feet of strike-slip displacement (Reference 19). The age of last movement on the fault is believed to be late Paleozoic (Reference 19).

The Royal Center Fault, which is located in northern Indiana about 150 miles southeast of the site, trends northeast-southwest for about 47 miles with the southeast side down about 100 feet relative to the northwest side (Reference 20).

2.5.1.1.4.2.7 Cryptovolcanic or Astrobleme Structures

There are four small cryptovolcanic or astrobleme structures within the regional area. These structures include the Des Plaines Disturbance and Glasford Structure in Illinois, the Glover Bluff Disturbance in Wisconsin, and the Kentland Disturbance in Indiana (Figure 2.5-13). These structures are all probably late Paleozoic or Mesozoic in age (Reference 21, Reference 22, and Reference 23).

2.5.1.1.4.2.8 Minor Faults Within 12 Miles of the Byron Site

A list of minor faults within 12 miles of the site is presented in Table 2.5-3 and the distribution of reported minor faults within 12 miles of the site is shown in Figure 2.5-16. A minor fault is defined in this report as having a surface trace of less than 1000 feet and a major fault, as having a surface trace of more than 1000 feet. Field examination by the Illinois State Geological Survey and by Dames & Moore personnel of all known faults within a 5-mile radius of the plant site shows no evidence of displacement within the Pleistocene deposits and no evidence of offset at the bedrock surface. Minor displacement faults discovered during the excavation of the power block are described and discussed in FSAR Attachment 2.5C and are presented in Table 2.5-3.

2.5.1.1.4.2.9 Age of Faulting in Northern Illinois

The age of the faults in northern Illinois is based on the Illinois State Geological Survey and Dames & Moore's investigations. These investigations show that the Pleistocene deposits are not offset over the faults and there are no thicker accumulations of Pleistocene deposits on the downthrown side which indicates that the bedrock surface on the upthrown side has been eroded before Pleistocene deposition. In addition, drainage patterns in the area that cross the faults show no cutoff along the fault trace. On the basis of this information which has all been visually field checked by the Illinois State Geological Survey and Dames & Moore, the age of the faulting generally is established as pre-Pleistocene. The geological history as interpreted from the various geological studies indicates that the last major tectonic activity occurred in the post-Pennsylvanian pre-Cretaceous period; therefore, the faulting is assumed to have occurred during this time interval. This age of the faults makes the faults noncapable as defined by 10 CFR 100, Appendix A, "Seismic and Geologic Siting Criteria."

2.5.1.1.4.2.10 Faults Beyond 200 Miles from the Byron Site

Beyond 200 miles, but of importance to the regional geology are the fault zones in the southern Illinois area.

2.5.1.1.4.2.10.1 Rough Creek Fault Zone

The Rough Creek Fault Zone trends east-west across southern Illinois into Kentucky (Figure 2.5-13). The western portion of this fault zone has previously been referred to in the literature as the Cottage Grove Fault Zone and the eastern portion of this fault zone has previously been referred to in the literature as the Shawneetown Fault Zone. Good exposures along the fault zone are rare and most interpretations are based on subsurface data. According to Bristol and Buschbach (Reference 24, p. 27) and Sutton (Reference 25, pp. 69 and 70), the fault zone consists at some localities of a series of high-angle reverse faults with the south side being the upthrown side and at other localities of a series of normal, block faults. Summerson (Reference 26) and Heyl (Reference 27, p. 885) suggest strike-slip or wrench-type movement for this system. Heyl states that the numerous horsts and grabens are typical of wrench-type faults. Details of the faulting in the Rough Creek area are shown in Weller, Grogan and Tippie (Reference 28), Stonehouse and Wilson (Reference 29), Heyl (Reference 27), and in the Background Materials for and the proceedings of Symposium on Future Petroleum Potential of NPC Region 9 (References 30 and 31).

2.5.1.1.4.2.10.2 Structural Relations of Faults North and South of the Rough Creek Fault Zone (Including the Wabash Valley Fault Zone)

Faulting is present both north and south of the Rough Creek Fault Zone. The faults on the north side strike northeast and those along the Illinois-Indiana border are collectively referred to as the Wabash Valley Fault Zone (Figure 2.5-13). These northeast trending faults, including the Wabash Valley Fault Zone, are high-angle faults with maximum displacements within the magnitude of 200 to 300 feet. The location and northern extent of these faults is well defined on the basis of boring data. These faults terminate at the Rough Creek Fault Zone and are offset from the traces of the faults on the south side of the Rough Creek Fault Zone. The lack of continuity of the faults north and south of the Rough Creek Fault Zone is further suggested by an approximate 50 miles of right lateral displacement of the basement along the Rough Creek Fault Zone in eastern Kentucky (Reference 32). On his small scale map, Heyl (Reference 32) has the southern faults terminating as mentioned above. Dames & Moore in its investigation of these faults has contacted members of the State Geological Surveys and the United States Geological Survey who have studied this fault zone and have incorporated their recommendations and conclusions.

The number of faults and their displacements is much greater on the south side of the Rough Creek Fault Zone than on the north side. The faults on the south side of the Rough Creek Fault Zone trend north-east and east-west, and displacements in excess of 1500 feet have been reported by Eardley (Reference 21, p. 254). The faults terminate on the south side of the Rough Creek Fault Zone.

The relationship of faults north and south of the Rough Creek Fault Zone to the regional seismicity is given in Subsection 2.5.2.3.1.

A fault zone along the Mississippi Valley is located in southern Illinois and in adjoining states, approximately 300 to 350 miles south of the plant site. In southern Illinois, this structure consists of a swarm of northeast striking faults. The geological and geophysical evidence suggests that the Mississippi Valley Fault Zone is associated with the Mississippi Embayment tectonic element.

2.4.1.1.4.2.10.3 Ste. Genevieve Fault Zone

At the western end of the Rough Creek Fault Zone there are a series of northwest trending faults along the border between southern Illinois and southeast Missouri, several of which are grouped as the Ste. Genevieve Fault Zone (Figure 2.5-13). These faults are generally high angle faults along which the north side has moved down relative to its south side. Displacements of 1000 to 2000 feet have been reported (Reference 11).

2.5.1.1.4.2.10.4 Age of Faulting in Southern Illinois and Adjacent Areas

The age of the faulting in southern Illinois, southern Indiana, northern Kentucky, and southeastern Missouri is post-Pennsylvanian to pre-Pleistocene (Reference 11). In southern Illinois and northern Kentucky, Cretaceous age deposits are not cut by the faults and it is possible that all the faulting in this area is pre-Cretaceous in age. Willman (Reference 33, geologic map and Section A-A) and NPC Region 9 Symposium Proceedings (1971) show unfaulted Cretaceous deposits overlying the faults on the south side of the Rough Creek Fault Zone.

Ross (Reference 34) states that faults in the upper portion of the Mississippi Embayment area may still be active. This was discussed at great length with members of the Illinois State Geological Survey, Indiana Geological Survey, and U. S. Geological Survey. The conclusions are that there is no evidence of displacement of Pleistocene deposits associated with the Rough Creek Fault Zone or with the faults on either side of this zone and there is no evidence of displacement of the Cretaceous sediments in southern Illinois or northern Kentucky. The statement of no displacement of Cretaceous sediments is based on information presented in Graton and Sales (Reference 35, pp. 376-379). It was further stated that in Illinois the term "active" has been loosely applied to faults indicating areas of seismic activity rather than surface movements along a fault plane. Grohskopf (Reference 36, pp. 26-28) states that faulting of Pliocene gravels and Pleistocene loess is present in southeastern Missouri. However, the upper Cretaceous rocks of the Gulf Coastal Embayment area clearly overlap the intensely faulted area in southern Illinois, and only a few minor faults, possibly the result of slumping or solution-collapse, have been found cutting the Cretaceous, Tertiary, or Pleistocene rocks (Reference 3). In all cases, alternative explanations are possible to explain the displacement of these deposits.

2.5.1.1.5 Gravity and Magnetic Anomalies

Measurements of the earth's gravitational and magnetic fields have been made both on the ground and from the air in Illinois and surrounding areas. The results can be interpreted in terms of past and present structural activity.

Gravity anomalies are usually caused by some combination of three major factors: (1) structure, unconformities and lithologic changes in the sedimentary rocks; (2) relief on the crystalline basement surface; and (3) lateral density changes in the crystalline portion of the earth's crust and upper mantles (Heigold, Reference 37).

Figure 2.5-17 represents the Bouguer Gravity Anomaly Map of the region surrounding the site. Some of the anomalies appear to be associated with some of the regional geological structures.

An anomaly trend in east-central Illinois appears to follow the trend of the LaSalle Anticlinal Belt. In south-central Illinois a gravity anomaly appears to coincide with the position of the Illinois Basin. In northwest Illinois a general east-west trending anomaly appears to follow the trend of the Plum River Fault Zone, and in north-central Illinois an anomaly appears to coincide with the position of the Ashton Arch, on the southwest side of the Sandwich Fault Zone. The relation of gravity anomalies to other geologic structures is not apparent on the regional basis.

The regional magnetic anomaly map is shown in Figure 2.5-18. It shows a magnetic anomaly on the south side of the Sandwich Fault Zone suggesting either the basement rock is closer to the surface than in adjacent areas or that there is a change in the magnetic susceptibility of the rocks. The trends of the Wisconsin Arch, the LaSalle Anticlinal Belt, and the Kankakee Arch are weakly delineated. The magnetic maps for the surrounding states are regional maps and have a larger contour interval than Figure 2.5-18, structural trends and lineations are very difficult to delineate.

2.5.1.1.6 Man's Activities

For a discussion of man's activities in the site area, refer to Subsection 2.5.1.2.7.

2.5.1.2 Site Geology

2.5.1.2.1 General

The site is located in an upland position about 2 miles east of the Rock River (Figure 2.5-2). The surface elevation at the site ranges between 722 and 893 feet. This is approximately 52 to 223 feet above the Rock River. The topography, shown on Figure 2.5-2, is that of a dissected undulating plain with a well developed drainage pattern.

Unconsolidated sediments in the site area, which are mainly Pleistocene in age, include alluvium, loess, till (with minor amounts of outwash silts, sands, and gravels), and residuum. The majority of these materials were deposited during the Wisconsinan glacial stage. The combined deposits of till, loess, and residuum reach a maximum thickness of 37 feet in the site area. Boring G-23, drilled in the valley of the Rock River at the site of the river screen house (Figures 2.5-2 and 2.5-5), penetrated 113 feet of alluvial material overlying the St. Peter Sandstone. As a result of post-glacial erosion, overburden is commonly thin or absent along drainageways to the Rock River.

The unconsolidated deposits overlie approximately 2500 to 3000 feet of Ordovician and Cambrian sedimentary strata which are in turn underlain by a Precambrian igneous basement rock complex. Figure 2.5-19 shows the stratigraphy of the site area, and Figure 2.5-20 shows bedrock topography and bedrock geology of the site area.

2.5.1.2.2 Physiographic Setting

The site is located within the Rock River Hill Country subsection of the Till Plains Section of the Central Lowland Physiographic Province (Reference 38). The Till Plains section is characterized by a widespread mantle of glacial drift deposited over an irregular bedrock erosion surface. The thickness of the drift varies widely depending on an number of factors including the nature of the bedrock topography, the glacial history, and the amount of post-glacial erosion.

Deposits from Wisconsinan and Illinoian stages are included in the total thickness of drift. The glacial stages have been delineated largely by the distinctive physical properties, texture, and mineral composition of the various tills (Reference 39). The character and composition of the drift is highly variable and depends on the age and the origin of a particular deposit. Overburden deposits of glacial tills, outwash, or lacustrine sediments are most common and the deposits vary greatly in both horizontal and vertical extent. Boundaries are often gradational or poorly defined. The Till Plains Section in the site area was blanketed by 3 to 6 feet of Wisconsinan loess.

Post-glacial erosion has produced a dissected topography in this region. Erosion along the existing creeks and gullies, tributaries of the Rock River (2 miles west of the site), has locally removed the loess and till, thereby exposing the underlying bedrock.

General physiographic features in the site and immediate surroundings (Figure 2.5-21) include bottomlands and floodplains, moderately to steeply sloping uplands and valley walls, and level to gently rolling uplands. The topography in the northern third of the site area is dissected by Woodland Creek which drains northwestward to the Rock River. The topography in the southern half of the area is slightly dissected and rolling and slopes to the southwest. A level to gently rolling northwest-trending upland ridge extends through Section 13 and merges with a broader north-south ridge which parallels the site on the east. Elevations range from 893 feet in the southeastern portion of the site to 722 feet in the northwestern corner of the site area. The elevation of the site area ranges from approximately 52 to 223 feet above the level of the Rock River.

Scattered small depressions, or sinkholes, are present in the plant site area. These sinkholes, shown on Figure 2.5-21, have been caused by solution of dolomitic bedrock. Four "sinkholes" or solution basins identified in Figure 2.5-101 were located from field reconnaissance and inspection of aerial photographs of the site area. The dimensions of the solution basins were determined from aerial photographs of the site area and from field measurements. Forty-one borings and three test pits were made in 1973 along the Essential Service Water Makeup line to determine the details of subsurface strata. The borings ranged in depth from 5 to 115 feet below the ground surface and were drilled at the locations indicated in Figure 2.5-6. Two seismic refraction lines were made during 1972 in the area of the largest solution to determine subsurface conditions (Figure 2.5-63). These field efforts confirmed the lateral dimensions and extent of the solution basins as determined from aerial photograph measurements. One boring, SH-1 shown in Figure 2.5-102 was drilled in the center of the large solution basin to determine the vertical extent of the feature. There is no evidence of recent solution basin activity.

One solution basin is present in the site area with a diameter greater than 50 feet and it is the only basin for which it was possible to determine a relative age. This solution basin is 150 feet in diameter and has 31 feet of Wisconsinan and residual soil deposits overlying the bedrock surface at its center. The feature was dated as Wisconsinan to pre-Pleistocene in age based on the soil deposits within the depression. This solution basin is at least 7,000 years old and possibly more than 2 million years old (Wisconsinan to pre-Pleistocene). The dimensions of this solution basin are partially attributable to continued solution activity during the Pleistocene Epoch. In addition, climatic conditions and periglacial effects associated with glacial advances that are not present today or predicted during the life of the plant also have contributed to its size. Solution basin dimensions average approximately 50 feet in diameter and reach a maximum diameter of about 150 feet. All the other solution basins are located in areas where there are little or no Wisconsinan and residual soil deposits, such as along the bluff area west of the plant. These smaller sinks are undated and may be Holocene in age. Therefore, any sinks that could form during the life of the plant will be similar to these Holocene (?) age features and 50-foot diameter was assumed for design. Rock below major foundations has been grouted to seal possible subsurface solution features (Subsection 2.5.4.12). Solution activity is discussed in greater detail in Subsection 2.5.1.2.6.

2.5.1.2.3 Stratigraphy

The stratigraphy at the Byron site is well known, both from published material and from numerous borings completed during the course of this investigation. The excavation mapping program confirmed the interpretation of the subsurface geology discussed in the Byron PSAR.

In general, the site is underlain by a regular sequence of units marked by a remarkable uniformity and continuity. A detailed discussion of stratigraphy follows.

2.5.1.2.3.1 Soil Deposits

2.5.1.2.3.1.1 General

The soil deposits in the site area which were observed during the drilling program and construction of the Byron Station site are identical to those predicted for this portion of the regional area (Subsection 2.5.1.1.3.1). These soil deposits include alluvium, loess, and till of Pleistocene age, and residuum. The residuum has been dated as being at least Yarmouthian in age, and possibly as old as Tertiary in age (FSAR Attachment 2.5C). These soil deposits have been differentiated into formations and members based on the established practice of the Illinois State Geological Survey. These stratigraphic units are discussed in Subsections 2.5.1.2.3.1.2 and 2.5.1.2.3.1.3. The sequence and nature of the soil and rock units in the site area are shown in a composite stratigraphic column, Figure 2.5-19 and in subsurface sections on Figures 2.5-59 through 2.5-61.

Borings at the site vicinity encountered soil deposits which ranged in thickness from 4 feet at Boring G-12 (Figure 2.5-119), to 37 feet at Boring G-25 (Figure 2.5-140). The average thickness at the plant location is about 9 feet. Soil thickness at the plant location is shown on Figure 2.5-23.

2.5.1.2.3.1.2 Pleistocene

Alluvial deposits are located along Woodland Creek and along the Rock River. The alluvial deposits of Woodland Creek are from 50 to 100 feet wide and probably average about 15 feet in thickness, becoming thicker downstream of the plant site. The upper deposits are brown, medium-stiff clayey silts; below the clayey silts are reddish-brown, medium stiff to very stiff clayey sands. The sands are considered to have relatively high permeabilities.

Along the Rock River the alluvial deposits are from 0.5 to 1 mile wide. Boring G-23 at the site of the river screen house (Figure 2.5-5) penetrated 113 feet of alluvial material overlying the St. Peter Formation. The alluvial deposits along the Rock River consist of medium-dense to very dense, well graded, brown, gravelly fine to coarse sand. Cobbles grade in at 95 feet. These materials are very permeable and groundwater was encountered at 4 feet.

The uppermost soil deposit in the upland areas is undifferentiated Peoria Loess. The Peoria Loess is a windblown silt, generally less than 3 feet in thickness at the plant site, that mantles various glacial deposits of Wisconsinan or Illinoian age.

Three glacial till members have been identified at the plant site. These till deposits are not consistently present throughout the site, having irregular thicknesses and distributions. Furthermore, at no one place at the site are all Quaternary units present in a complete stratigraphic section. The youngest till member is the Esmond Till Member which was originally interpreted as a member of the Wedron Formation. The Esmond Till Member is generally a medium stiff to stiff, brown silty clay to clayey silt till which contains scattered boulders. The unit was deposited during the Woodfordian Substage of the Wisconsinan Stage. The Morton Loess separates the Esmond Till Member from the underlying Argyle Till Member. The Morton Loess is generally a stiff, brown to gray, calcareous silt. The Argyle Till Member is a sandy, generally medium stiff to stiff, brown till that was deposited during the Altonian Substage of the Wisconsinan Stage. The Argyle Till Member is separated from the underlying Ogle Till Member by sediments that are in part stream deposited, and in part lacustrine in origin. These sediments were apparently deposited in front of the advancing Wisconsinan glacier. The oldest glacial till at the site is the Ogle Till Member of the Glasford Formation and is considered to be Illinoian in age. This unit was observed in the trench wall at the solution basin discussed in the Dames & Moore Report, "Geologic Investigation of Solution Features," dated January 29, 1982. In the western portions of the site, where the Argyle is present, it directly overlies weathered bedrock or residual soil (Byron Station Subsection 2.5.1.2.3.1.2). The Esmond Till is present only in the extreme eastern edge of the site and is not present west of the cooling towers.

Unpublished studies performed in 1982 by the Illinois Stage Geological Survey (ISGS) reinterpret the age of the Esmond and Argyle tills. These studies now show that the Esmond correlates with the Sterling or Radnor tills of the Illinoian Stage. Furthermore, these units have been stratigraphically miscorrelated in the past and will be redesignated in future nomenclature (Drs. John Kempton and Leon Follmer, ISGS, personal communication). The present understanding of glacial stratigraphy in northern Illinois shows that no Wisconsinan tills occur at the site. The nearest known Wisconsinan till is found in southeastern Ogle County in the Bloomington Moraine. At the plant site the Ogle Till Member is a sandy till, containing more than 70% sand and less than 10% clay.

2.5.1.2.3.1.3 Pleistocene-post-Ordovician Residuum

The Pleistocene-post-Ordovician soil deposits underlying the Pleistocene glacial deposits at the Byron Station site consist of highly weathered bedrock overlain by a residual soil. The highly weathered bedrock is generally found throughout the site area. It consists of a loose to very dense, yellow, silty dolomitic sand with some dolomite gravel (weathered rock fragments) in grain to grain contact. This material contains Ordovician age siliceous fossils, suggesting that the soil's parent material was from the Galena Group. In the site borings, this material ranged in thickness from 0 to a maximum of 5-1/2 feet in Boring G-3 (Fiqure 2.5-108). As exposed in the trenches for the Fault

Specific Geotechnical Investigation, this material ranged in thickness from 1 to 3 feet (FSAR Attachment 2.5C).

Overlying the highly weathered bedrock is a residual soil. This material is a highly weathered, reddish-brown clay. The residual soil also contained Ordovician age siliceous fossils, suggesting that the soil's parent material was from the Galena Group (FSAR Attachment 2.5C). The residual soil has numerous pendants which extend down former joint surfaces and indicate development on a highly developed weathered surface. The residual soil is only locally present, as it has been mainly removed by glacial erosion during the advancement of Illinoian or older glaciers. Where present, the residual soil is 0.5 to 1.0 feet thick (FSAR Attachment 2.5C).

According to Willman (FSAR Attachment 2.5C), the residual soil at the site is similar to other residual soils throughout the state. The composition of the clay samples taken in the west wall of the main excavation is essentially the same as those discussed by Willman and Frye (Reference 40). This residual soil is Yarmouthian or older in age and is most likely Tertiary in age on the basis of mineralogical and physical similarity to other Tertiary age soils.

2.5.1.2.3.2 Bedrock Deposits

The bedrock deposits encountered in the site vicinity correlate with those of the regional area (compare Figures 2.5-8 and 2.5-19). The bedrock deposits in the vicinity of the site range in age from Ordovician to Precambrian as shown in Figures 2.5-8 and 2.5-19. The deepest boring completed at the Byron Station site extended into the Ordovician-age St. Peter Sandstone. The excavation for the main plant structures extended into the uppermost bedrock formation, the Dunleith Formation. The condition of the Dunleith Formation (jointing, faulting, etc.) as exposed in the main plant excavation is discussed in FSAR Attachments 2.5C and 2.5D. The two plant water wells partially penetrated the Cambrian age Mt. Simon Sandstone. The wells are described in UFSAR Subsection 2.4.13.1.3.

The descriptions of pre-St. Peter units were obtained from published data in the immediate area of the site. Thicknesses of the pre-St. Peter units were obtained from isopach work-maps on file at the Illinois State Geological Survey. The presence of all units from the top of the Shakopee Formation to the top of the Potosi Formation has not been verified at the site location. These units are present in the area south of the site where the St. Peter rests unconformably on the Shakopee Formation. In the area north of the site, pre-St. Peter erosion has completely removed the Shakopee through the Potosi Formations and the St. Peter rests unconformably on the Franconia Formation. The plant water wells indicated that the Shakopee Formation through the Eminence Formation have been locally eroded and removed from the site area.

Throughout the site the general lithological characteristics of the formations encountered remained quite constant, as indicated on the generalized geologic sections (Figure 2.5-24). However, the quality of the rock units and their engineering characteristics are affected by the presence of jointing. In the upland regions and away from major joint patterns, the formations are good to excellent in quality based on RQD values (Table 2.5.4). Near major joint patterns or joint controlled drainageways, the rock quality decreases as a result of weathering along the joints. The formations within the Galena and Platteville Groups have been locally subjected to solution activity resulting from the movement of groundwater both along bedding planes and along major joints. Highly weathered open bedding planes were inferred from the rock cores and are described on the boring logs. These may be partially sand or clay filled and are on the order of 1/8 inch wide.

Core recovery was determined by measuring the actual amount of rock core recovered in each run. This amount is then expressed as a percentage of the total length of the core run. Void areas were included in the determination of the core recovery and are reflected in the maximum, minimum, and average values for core recovery.

Rock quality designation (RQD) has been used extensively as an indication of the general quality of the rock. This procedure employs a modified core recovery percentage in which only the pieces of sound core 4 inches and longer are counted as recovery. This modified sum is then expressed as a percentage of the total length of the core run.

The core from the initial geological borings (the G series) was obtained using a NX double tube core barrel. The plant location borings (the P series) were done using a NX wireline core barrel. The RQD and recovery values were much higher where the wireline drilling techniques were utilized; therefore, the differences in RQD and core recovery values obtained by the two different techniques do not necessarily reflect changes in subsurface conditions but may be indicative of differences in drilling techniques. It is likely that the wireline drilling technique also did not disturb the boring walls as much as the conventional drilling techniques; therefore, the results of water pressure tests from two borings drilled by two different techniques cannot be directly correlated. However, a comparison of the results does provide a relative indication of zones of water losses; i.e., a given zone has a higher water loss than the zone above and below.

The degree of weathering was classified visually according to Table 2.5-5. In the plant location, the core from Borings P-2 and P-3 were photographed to show visual characteristics of the rock encountered. These photographs are shown in Figure 2.5-25.

The results of the Seismic Refraction Survey indicate that the compressional wave velocities vary both vertically and horizontally. With the exception of the soil-bedrock interface, the horizontal and vertical variations in most cases appear to be primarily an indication of the degree of weathering and fracturing the rock has undergone, with the higher velocities generally indicating more competent rock. Within the plant site area, the vertical velocity interfaces within the bedrock correlate with lithology.

2.5.1.2.3.2.1 Galena Group

2.5.1.2.3.2.1.1 Dunleith Formation

This formation is a buff, fine to medium crystalline, thin to massive bedded dolomite which contains zones of chert nodules. Green shale partings occur in its lower portions. The Dunleith Formation is the upper bedrock unit at the site and provides topographic control, forming flat to gently rolling upland. Outcrops of this formation are found in the slopes of Woodland Creek.

At the plant location contours on the bedrock surface indicate that the bedrock topography generally follows surface topography with a northeast trending bedrock high in the center of the plant location (see Figure 2.5-7 and Figure 2.5-26). Bedrock elevation at the plant location ranged from a high of 876.3 in Boring P-31 to a low of 850.3 in Boring P-9.

The maximum thickness of Dunleith encountered was 105 feet in Boring G-10. At the plant location the average thickness was 90 feet, ranging from 102.2 feet in Boring P-31 to 70 feet in Boring P-9.

This unit is slightly to moderately weathered in all borings because of its surface exposure. Solution activity has occurred along many of the joints, fractures and bedding planes, and reddish-brown clays or yellow silty sands may be found along these planes. Occasional small sinkholes are developed in this formation. Because of the mantle of till these sinkholes are subdued but they appear to have a maximum diameter of 150 feet, and from surface observation and geophysical data, they appear to extend downward to the base of the Dunleith; however, they may extend deeper. Descriptions of the Dunleith Formations as exposed in the excavation for the main plant structures are given in FSAR Attachment 2.5D.

The Dunleith Formation contains zones of chert nodules, which in the plant location are concentrated from elevation 817 to elevation 804. Thin green shale partings are predominant in the lower portions of the Dunleith and in the plant location these shale partings grade in at approximate elevation 804. RQD values for the geologic borings (G series) ranged from 0% to 93%, and core recovery values ranged from 0% to 100%. RQD values from the plant location borings (P series) ranged from 82% for Boring P-34 to 26% for cores from P-9, P-21, and P-25 with an average of 57%, and core recovery values ranged from 70% for Boring P-21 to 100% for many borings with an average of 97%. The high core recovery values and generally low RQD values in the plant area indicate that the Dunleith is fractured, jointed and thin bedded, but is not subject to the presence of large openings along joints and bedding planes. The variations at the plant location result from both vertical variations in lithology and the location of the borings relative to a major joint. From the top of the rock to approximate elevation 817 feet, RQD values average 58%. From elevation 817 feet to 804 feet, the RQD values average 44%. This decrease in RQD values is caused by fracturing of the rock during drilling due to the high chert content of this zone. Excavation mapping supports this statement. From elevation 804 to the base of the Dunleith the RQD values averaged 60%. The zones of low RQD for all three zones follow weathered joint patterns.

The results of water pressure test data are affected by rock quality: i.e., zones which have low RQD values usually exhibit higher water losses, but other factors must also be considered. RQD, open fractures, open bedding planes, and degree of weathering are all important in evaluating amount of water loss, and these are directly related not only to the proximity of major joints but also to the path groundwater takes to reach these joints. The combination of these factors resulted in generally higher water losses in the northern one-third of the plant location and in a zone in the south central part of the plant location. Table 2.5-6 lists the holes tested in the plant location and the average water losses in water loss units for the Dunleith Formation. A water loss unit is defined in Subsection 2.5.4.3.2.1.

Laboratory testing of 16 samples from the Dunleith Formation showed a large variation in the unconfined compressive strength with a maximum value of 17,700 psi from Boring G-12 to a minimum of 2,900 psi from Boring P-7. Rock samples from the plant locations showed variations from 2,900 psi for a moderately weathered sample from Boring P-7 to 14,300 psi for a slightly weathered sample from Boring P-4. The results of geophysical logging showed the Dunleith to be a weathered, fractured dolomite. The logs (mainly the 3-Dimensional Borehole Log) also indicate an increase in fracturing through the zones high in chert and an increase in argillaceous material below the chert zones, substantiating the results from core logs.

Test data from two samples from Boring G-12 resulted in compressional wave velocities (shockscope) ranging from 17,500 to 18,100 ft/sec. These values are higher than those obtained during geophysical exploration, Subsection 2.5.4.4, which gave the Dunleith Formation compressional wave velocities ranging from approximately 7,500 ft/sec to 11,000 ft/sec.

Resonant column tests were performed on two samples from Borings P-2 and P-3 of the Dunleith Formation in order to evaluate the shear wave velocity and modulus of rigidity. The test results indicated a range in the shear wave velocity of 3000 to 3650 ft/sec and in the modulus of rigidity of 45×10^6 to 69×10^6 psf.

2.5.1.2.3.2.1.2 Guttenburg Formation

The Guttenburg Formation is a buff and gray, finely crystalline, thin to medium-bedded dolomite with reddish-brown and gray shale partings. The reddish-brown shale partings are quite distinctive and help make this formation a marker horizon both in cores and on geophysical logs. The formation ranged in thickness from 6.8 feet at Boring P-19 to 3.5 feet in Borings G-17, G-19, G-22, G-5, and G-8. The average thickness from 45 borings in the plant location was 5.5 feet. This unit may be exposed at the lower elevations in the northwestern portion of the site area (Figure 2.5-20).

In the site area RQD values for the Guttenburg range from 0 to 91% and the core recovery ranged from 9 to 100%. At the plant location core recovery ranged from a low of 66% from Boring P-21 to 100% in may borings. RQD values ranged from 14% in Boring P-21 to 85% in Boring P-36 with an average from 45 borings of 59%. Geophysical borehole logging showed that this formation consists of dense to fractured dolomite, which agrees with the range in RQD values from the cores. At the plant location the top of the Guttenburg ranged from elevation 766.9 in Boring P-4 to 781.7 in Boring P-19 which is an average depth of approximately 100 feet below the ground surface.

Water pressure tests were performed with a 10-foot packer spacing; therefore, the test results were not limited to this formation. Water losses were moderate.

Five samples of the Guttenburg in the site area were subjected to unconfined compression tests. The results from these tests ranged from a minimum of 6,100 psi in Boring G-5 to a maximum of 14,400 psi in Boring G-4. In the plant location one sample from Boring P-13 was tested and resulted in a value of 7,500 psi.

A compressional wave velocity (Shockscope) of 15,600 ft/sec was obtained from one sample of the Guttenburg from Boring G-12. This value is in good agreement with the results of field geophysical testing which gave compressional wave velocities ranging from approximately 9,500 ft/sec to 15,250 ft/sec.

Resonant column tests were performed on one sample from Boring P-3. The test results indicate a shear wave velocity in the range of 3000 to 3400 ft/sec with the modulus of rigidity ranging from 47×10^6 to 60×10^6 psf.

2.5.1.2.3.2.2 Platteville Group

2.5.1.2.3.2.2.1 Quimbys Mill Formation

The Quimbys Mill Formation is a buff and gray, finely crystalline, thin to medium-bedded dolomite with dark gray shale partings and occasional gray and white chert nodules. In the site area the formation ranged in thickness from 14.6 feet at Boring P-37 to 7 feet at Boring G-8. At the plant location the formation ranged in thickness from 9.6 feet in Boring P-5 to 14.6 feet at Boring P-37, with an average thickness from 45 borings of 11.7 feet. The elevation of the top of the Quimbys Mill at the plant location ranged from 775.8 feet in Borings P-19 and P-23 to 761.6 feet in Boring P-21. At the plant location, the Quimbys Mill occurs at an approximate depth of 105 feet below the ground surface and it may be exposed on some hillsides and creek bottoms in the northwest part of the site.

Throughout the site area the rock quality of the Quimbys Mill varies considerably. In areas away from major creeks and weathered joints, RQD values ranged from 68% to 90% with core recovery values up to 100% (Borings G-5, G-12, and G-20, Figures 2.5-110, 2.5-119, and 2.5-133). Near joints RQD values were as low as 0% and core recovery values were as low as 9% (Boring G-21, Figure 2.5-134)

At the plant location this formation was generally quite competent with many borings characterized by 100% recovery. Core recovery at the plant location averaged 99% with a low of 91% in Boring P-28. RQD values average 71%, ranging from a low of 45% in Borings P-21 and P-27 to a high of 94% in Boring P-2. Zones of low RQD values generally follow joint patterns. Core recovery, RQD values and core descriptions show that the Quimbys Mill Formation is a fairly competent rock stratum throughout the plant location. This is also demonstrated on the geophysical borehole logs at the plant location which characterize this formation as dense, with slight fracturing but free from any major fracturing. Small voids less than 3 inches in diameter may be present as in Boring P-28.

In the site area water pressure test data for the Quimbys Mill showed wide variation. Highest water losses were reported in Boring G-16 which is located near Woodland Creek.

At the plant location water losses in the Quimbys Mill were generally quite low as is shown on Borings P-2 and P-3, Figures 2.5-142 and 2.5-143, where losses were less than 1.7×10^{-3} water loss units. Boring P-22, near a weathered joint, resulted in the highest loss for the Quimbys Mill Formation.

Seven samples of the Quimbys Mill Formation were subjected to unconfined compression tests. The results showed the Quimbys Mill to have the highest strength at the plant location (17,700 psi from Boring P-6 and 19,100 psi from Boring P-16). In the site area, the lowest strength 7600 psi, was obtained from a sample from Boring G-13.

Borehole geophysical data resulted in compressional wave velocities for the Quimbys Mill Formation which ranged from 12,000 to 15,500 ft/sec and shear wave velocities which ranged from 4,500 to 9.500 ft/sec.

2.5.1.2.3.2.2.2 Nachusa Formation

This formation is a mottled gray and buff finely crystalline, medium to massive-bedded dolomite with thin gray shale partings and occasional chert nodules. In the site area the formation ranged in thickness from 12.5 feet in Boring G-12 to 24 feet in Boring G-15. At the plant location this formation ranged in thickness from 13.5 feet in Boring P-4 to 19.5 feet in Boring P-9 with an average thickness from 45 boring of 16.5 feet. This formation may be exposed on some hillsides and creek bottoms in the northwest area of the site (Figure 2.5-20). At the plant location the elevation of the top of the Nachusa Formation ranged from 763.8 feet in Boring P-9 to 751.5 feet in Boring P-21 at an approximate average depth of 118 feet below the surface.

In the site area RQD values were generally greater than 80% with core recovery values ranging from 90% to 100%. Scattered high angle fractures were present (Borings G-2, G-6, and G-13, Figures 2.5-106, 2.5-111, and 2.5-121). Near major joints a few small voids (3 to 6 inches in diameter) are present and RQD values of 20% and core recovery values of 40% were recorded (Boring G-21, Figure 2.5-134).

At the plant location core recovery averaged 93% with a low of 50% from Boring P-19. RQD values average 72% and ranged from 93% in Boring P-33 to 26% in Boring P-9. Small voids were encountered in some borings. The geophysical borehole logs and analyses of cores indicate that these voids are generally open but may contain some clay and sand (weathered dolomite). The geophysical logs showed that this formation is dense but may be highly weathered and fractured, especially in its lower parts. Zones of low core recovery and RQD values generally correlate with joint patterns.

In the site area water pressure testing resulted in water losses no greater than 10 lugeons in this formation (Borings G-6, G-8, and G-9, Figures 2.5-111, 2.5-114, and 2.5-116, and Subsection 2.5.4.3.2.1). At the plant location, unit values ranged from zero in Borings P-2 and P-3 located in the center of the plant location to greater than 1.4 $x10^{-2}$ water loss units in Boring P-22 located in the south central part of the plant location and Boring P-23 located in the northwest part of the plant location.

Unconfined compressive strengths of six samples of the Nachusa Formation ranged from 3,600 psi for a sample from Boring P-16 to 19,100 psi for a sample from Boring G-12.

Resonant column testing of one sample of the Nachusa from Boring P-2 indicated shear wave velocities for the Nachusa Formation in the range of 4341 to 4828 ft/sec with the modulus of rigidity ranging from 93 x 10^6 to 116 x 10^6 psf. The shear wave velocities obtained from resonant column testing are lower than the results from borehole geophysics which resulted in in situ compressional wave velocities of 15,500 ft/sec and shear wave velocities of 9,500 ft/sec.

2.5.1.2.3.2.2.3 Grand Detour Formation

The Grand Detour Formation is a gray and buff, finely crystalline, slightly fossiliferous, medium-to massive-bedded dolomite with gray-brown and reddish-brown shale partings. This formation commonly has gray mottling, widely scattered chert nodules, and random, high angle fractures. Borehole data shows that this formation ranges in thickness from 30 feet at Boring G-6 to 46.3 feet at Boring P-29. At the plant location the formation averages 41.7 feet in thickness with a minimum thickness of 36.2 feet in Boring P-38. In the proximity of the site the Grand Detour may be exposed at some of the lower elevations along creek valleys (Figure 2.5-20).

At the plant location the top of this formation ranged in elevation from 736.9 feet in Boring P-13 to 746.6 feet in Boring P-19. This is at an approximate average depth of 135 feet below the ground surface.

Where the Grand Detour Formation has been unaffected by weathering or solution in the site area, RQD values ranged from 50% to 83% and recovery values averaged 90% (Borings G-8, G-20, and G-22, Figures 2.5-114, 2.5-133, and 2.5-136). Near joints and drainageways RQD values and core recovery were much lower and sometimes both reached zero in highly fractured zones (Borings G-16 and G-21, Figures 2.5-126 and 2.5-134).

At the plant location core recovery averaged 91% with a low of 42% in Boring P-21. RQD averaged 70% and ranged from 93% in Borings P-34 and P-39 to 11% in Boring P-21. Zones of low RQD generally correspond to zones of low recovery, indicating that low RQD values are primarily the result of the presence of voids. These zones are generally in close proximity of weathered joints.

Near the plant location, the Grand Detour Formation contains numerous voids which ranged up to 6.5 feet in size in Boring P-9. Core analysis and the results of borehole geophysics indicated that these voids are usually open but may contain clay and/or sand filling.

Water-pressure testing in the Grand Detour resulted in water losses which ranged from very low in a competent zone of Boring G-12 to very high in an open fractured zone in the same hole. At the plant location most tests resulted in very high water losses for this formation (see Borings P-7 and P-22, Figures 2.5-147 and 2.5-162).

Five samples from the Grand Detour Formation were subjected to unconfined compression testing. The results ranged from a low of 3,500 psi for a sample from Boring P-6 to 16,000 psi for a sample from Boring P-7.

A compressional wave velocity (shockscope) of 12,500 ft/sec was obtained from a sample of the Grand Detour Formation from Boring G-12. Borehole geophysical data indicated that compressional wave velocities average 15,500 ft/sec and shear wave velocities average 9,500 ft/sec.

2.5.1.2.3.2.2.4 Mifflin Formation

The Mifflin Formation is a finely crystalline, thin-to medium-bedded, gray or buff dolomite with blue-gray, gray, green, or brown shale partings, rare occurrence of chert nodules, and zones of orange speckling. In the site area the Mifflin ranged in thickness from 13 feet at Borings G-2 and G-12 to 26 feet at Boring G-6. At the plant location this formation ranged from 14.6 feet in Boring P-1 to 23.4 feet in Boring P-10 with an average thickness from 45 borings of 19 feet.

At the plant location the elevation of the top of this formation ranged from 694 feet in Boring P-1 to 706 feet in Boring P-19. The top of the Mifflin is at an approximate depth of 175 feet below the ground surface.

In the site area the formation was characterized by high RQD values and core recovery (Boring G-22, Figure 2.5-136). High angle fractures were present and in some localities these fractures have been subjected to solution activity. Fractured rubble zones up to 10 feet thick with low RQD and core recovery values were encountered in Borings G-16 and G-21 (Figures 2.5-126 and 2.5-134). These borings are located in the vicinity of joints.

At the plant location the Mifflin Formation has been subjected to more solution activity along joints than any other formation. Core recovery averaged 78% with a low of 22% in Boring P-21, and RQD values averaged 56% ranging from a low of 4% in Boring P-21 to 100% in Boring P-19. These low values reflect the generally higher calcite content of this formation making this formation more susceptible to weathering activity than are the more dolomitic formations. Low core recovery RQD values generally follow zones of jointing.

The Mifflin Formation contains many voids ranging up to 16 feet in size in Boring P-9 which is near a weathered major joint. Borehole geophysics indicates that the rock quality in this formation varies from dense competent dolomite to very highly fractured dolomite with many voids. These voids appear mainly to be open but contain more clay and sand filling than joints in the formations above.

Water-pressure test results are available only for zones with high RQD and core recovery values. Water losses in the high quality rock were very low. Higher losses would be anticipated in zones of lower RQD and core recovery.

Laboratory testing of three samples from the Mifflin Formation resulted in unconfined compression strengths ranging from 7,100 psi from a sample from Boring P-13 to 12,900 psi for a sample from Boring P-6. A sample from Boring G-12 resulted in a laboratory compressional wave velocity (shockscope) of 19,500 ft/sec.

2.5.1.2.3.2.2.5 Pecatonica Formation

The Pecatonica Formation in the site area is a mottled white and black, finely crystalline dolomite with medium to massive bedding. This formation contains thin brown shale partings and may contain some brown to brownish-gray dolomite and dolomitic limestones. The Pecatonica contains some random, high angle fractures. Test borings showed thicknesses ranging from 14.5 feet at Boring G-15 to 31 feet at Borings G-2 and G-17.

At the plant location this formation ranged in thickness from 18.5 feet in Boring P-6 to 30 feet in Boring P-45 with an average from 45 borings of 24 feet. The top of this formation ranged from elevation 675 feet in Boring P-7 to 686 feet in Boring P-30 at an average depth of 194 feet below the surface.

In the site area RQD values for this formation ranged from 50% to 90% with core recovery ranging from 87% to 90%. In Boring G-16 the RQD dropped to 17% where a joint was encountered along which here had been some solution activity.

In the plant location core recovery averaged 90% with a low of 29% in Boring P-43. RQD ranged from 99% in Boring P-33 to 6% in Boring P-21. Two trends in rock quality were observed. Low core recovery and RQD values followed joint patterns and also showed a general decrease in rock quality to the south. In the northern half of the plant location core recovery averaged 94% and RQD 78%, while in the southern half of the plant location core recovery averaged 85% and RQD 61%. This formation contains numerous voids ranging up to 3.25 feet in size in Boring P-32. These voids are predominantly in the upper portions of the formation in the southern portion of the plant location. Geophysical borehole logging indicated that this formation consists generally of dense, fracture-free dolomite.

Water-pressure test data within the Pecatonica Formation in Borings G-8, P-7, P-2, and P-3 indicated relatively low water losses. Higher losses are expected in areas adjacent to weathered joints.

Two samples of the Pecatonica Formation were subjected to unconfined compression testing which gave results of 9,700 psi in Boring P-6 and 12,500 psi in Boring P-7.

2.5.1.2.3.2.3 Ancell Group

2.5.1.2.3.2.3.1 Glenwood Formation

2.5.1.2.3.2.3.1.1 Harmony Hill Shale Member

The Harmony Hill Shale Member is a pyritic, slightly silty to sandy, green and gray, thinly laminated shale unit. In the site area this member ranged in thickness from 1.5 feet at Borings G-1, G-2, G-15, and G-17 to 5 feet at Boring G-8. At the plant location the Harmony Hill ranged in thickness from 2.4 feet in P-18 to 4.3 feet in P-12 and P-26, with an average thickness from 45 borings of 3.14 feet. This member serves as a distinctive marker bed both in the core and on geophysical logs.

In the site area fresh core from the Harmony Hill was characterized by RQD values averaging about 60% and core recovery of 90% or better (Boring G-16, Figure 2.5-126).

In the plant location core recovery averaged 96% with a low of 26% from Boring P-43, and RQD averaged 72% ranging from 0% in P-43 to 100% in Boring P-12. RQD and core recovery values in the Harmony Hill must be evaluated with caution as the unit has a tendency to stick in the core barrel and damage frequently resulted during removal. The Harmony Hill shale also has a tendency to slake upon exposure to the air.

The elevation of the Harmony Hill Shale Member ranged from 666.8 in Boring P-6 to 650 in Boring P-8. It occurs at an approximate average depth of 218 feet below the ground surface.

Water-pressure tests in the Harmony Hill Shale Member indicated low water loss rates at pressures less than overburden pressure and increases in water loss at pressures near or exceeding overburden pressure. This suggests that the shale is subject to fracturing along bedding planes at higher pressures.

2.5.1.2.3.2.3.1.2 Daysville Dolomite Member

The Daysville Dolomite Member is composed of light greenish-gray, fine, grained, argillaceous dolomite interbedded in its lower portions with green, pyritic, dolomitic sandstone. The top of the Daysville Dolomite Member is marked by 6 to 8-inch brown sandy shale. In the site area the formation ranged in thickness from 17.5 feet at Borings G-6 and G-16 to 32 feet at Boring G-5. At the plant location the Daysville ranged in thickness from 19 feet in Boring P-2 to 20.5 feet in Boring P-1, with an average thickness from 45 borings of 20 feet.

The elevation of the top of the Daysville ranged from 663.4 feet in Boring P-6 to 647.0 feet in Boring P-8, at an approximate average depth of 222 feet below ground surface.

In the site area RQD values averaged approximately 60% and core recovery averaged approximately 90%. At the plant location core recovery average 98% with a low of 90% from Boring P-5. RQD values averaged 79% ranging from a low of 67% in Boring P-1 to a high of 92% in Boring P-11. Water-pressure tests of the Daysville were dependent upon the presence of sandstone. Zones with little sandstone and high carbonate resulted in generally low water losses while zones of high sandstone resulted in high water losses.

2.5.1.2.3.2.3.2 St. Peter Sandstone

The St. Peter Sandstone is a quartzose sandstone which is medium to fine-grained, poorly graded, poorly cemented, and friable. Shale and siltstones within the unit are rare; however, the upper portion of this formation does contain occasional thin green shale partings. The St. Peter Sandstone was encountered in 24 borings but the total thickness was penetrated only in the two plant water wells. The maximum penetrations into the unit was 51 feet (Boring G-8). The St. Peter Sandstone is estimated to be 100 to 200 feet thick throughout most of the northern two-thirds of Illinois (Buschbach, Reference 8, p. 51), but may be thicker locally. It was 425 to 450 feet thick at the water well locations.

At the plant location the elevation of the top of the St. Peter Sandstone ranged from 627 feet in Boring P-8 to 643.6 feet in Boring P-6, at an approximate average depth of 242 feet below the ground surface.

RQD values for the St. Peter Sandstone ranged from 0% to 97% with core recovery ranging from 0% to 100%. Pressure testing in this formation was inconclusive as the packers could not be sealed. Laboratory compressional wave velocity (shockscope) tests performed on three samples from Boring G-6 resulted in compressional wave velocities ranging from 5500 to 6500 ft/sec.

2.5.1.2.3.2.4 Prairie du Chien Group

No borings on the plant site were extended any deeper than the upper portion of the St. Peter Sandstone. Only the plant water wells were extended deeper than the St. Peter Sandstone. The description of these deeper units was obtained from published data pertaining to the immediate area of the site. Thicknesses were obtained from isopach work-maps on file at the Illinois State Geological Survey. The plant water wells indicated that the Prairie du Chien Group was locally removed from the site area by pre-St. Peter Sandstone erosion.

2.5.1.2.3.2.4.1 Shakopee Dolomite

The Shakopee Dolomite is primarily a very fine, light gray to light brown dolomite. It contains oolitic chert, some beds of medium-grained sandstone, light gray to green shale and dolomitic sandstone. It may contain lenses of massive algae structures up to 10 feet high. The Shakopee may be up to 67 feet thick in site area, but it is locally absent at the site.

2.5.1.2.3.2.4.2 New Richmond Sandstone

This formation is a buff, moderately sorted, rounded, friable, medium-grained sandstone with some interbedded, light-colored sandy dolomite. Its maximum thickness near the site is estimated to be 35 feet, but it was locally absent in the plant water wells.

2.5.1.2.3.2.4.3 Oneota Dolomite

The Oneota is a medium to coarse-grained, light gray to pink dolomite with minor amounts of sand and chert. This formation's maximum reported thickness is southeast of the plant site, in Kankakee County, where it is 250 feet. It was not encountered in the plant water wells.

2.5.1.2.3.2.4.4 Gunter Sandstone

This formation is a medium-grained friable, subrounded sandstone containing beds of light gray, fine-grained dolomite, and minor amounts of light green shale. If present in the site area, it is no more than 15 feet thick. The wells confirmed that it is locally absent at the site.

2.5.1.2.3.2.5 Cambrian Formations

2.5.1.2.3.2.5.1 Eminence Formation

The Eminence Formation is a light gray to light brown, sandy, fine to medium-grained dolomite with some oolitic chert, and thin beds of sandstone. If present in the area of the site, it is estimated to be no more than 50 feet thick. It was locally absent at the locations of the two water wells.

2.5.1.2.3.2.5.2 Potosi Dolomite

This formation is a finely crystalline, slightly argillaceous, brown to light gray dolomite. It is generally slightly glauconitic near the top and glauconitic and sandy near its base. Based on regional data, the Potosi is estimated to be 100 feet thick in the site area. However, the plant wells indicated that it is approximately 25 to 50 feet thick at the site.

2.5.1.2.3.2.5.3 Franconia Formation

This formation consists of a light gray to pink, fine-grained dolomitic sandstone that is usually glauconitic, silty and argillaceous. At the site, its estimated thickness is 100 feet. This thickness was verified by the plant water wells.

2.5.1.2.3.2.5.4 Ironton and Galesville Sandstones

These units are a medium-grained, poorly sorted, white sandstone with some course-grained sandstone near the top. They grade downward to a white to light buff, clean to slightly silty, fine-grained sandstone. In the project area the estimated thickness of these units is 150 feet. The actual thickness as measured in the water wells is approximately 105 feet. The Ironton and Galesville Sandstones are a major aquifer.

2.5.1.2.3.2.5.5 Eau Claire Formation

This formation consists of a variety of lithologies which in the plant site area may include fine-grained, well sorted sandstone, beds of shale, siltstone, and dolomite. All of the formation may be glauconitic.

Regionally, this formation ranges in thickness from 370 to 575 feet and in the site area is estimated to be 400 feet thick. This is consistent with observations in the water wells.

2.5.1.2.3.2.5.6 Mt. Simon Sandstone

This formation is a fine to coarse-grained, poorly sorted, friable sandstone which contains occasional small gravel. Reported thickness for the Mt. Simon ranges from 1200 to 2900 feet. In the site area, its estimated thickness is 1500 feet. The Mt. Simon Sandstone and the basal sandstones of the Eau Claire Formation comprise a major aquifer. The plant water wells are completed in this aquifer.

2.5.1.2.3.2.6 Precambrian

No holes have reached the Precambrian in Ogle County. Available data indicate that the basement rock should consist of pink to light gray granites and granodiorites. The estimated depth to the top of the Precambrian is about 2500 to 3000 feet (Bradbury and Atherton, Reference 2).

2.5.1.2.4 Structure

2.5.1.2.4.1 Jointing

Air photo interpretation and field reconnaissance were used to delineate the joint patterns. Figure 2.5-22 shows the joint patterns which were identified in the site area. They are: (1) a northwest trending pattern paralleling the regional structural trend, (2) a northeast pattern essentially perpendicular to the regional structure, and (3) both a north-south and an east-west pattern transverse to the structure. All of the joints were not identified owing to vegetation and soil cover.

Air photo interpretation indicates that the joint patterns are spaced approximately 200 to 500 feet apart. Examination of bedrock exposures along Woodland Creek, in the quarry NE 1/4, SE 1/4, NE 1/4 of Section 13, T.24N., R.10E. indicate that these four major patterns are detectable near the surface but that the near-surface joint spacing decreases to 1 foot or less. Near surface joints in the plant site are usually open to a width of 1/16 to 1/8 inch. Some of these joints are clean and some joints are clay filled. Examination of all available core indicates that while the degree of fracturing decreases with depth, there is no significant decrease within the Dunleith Formation above the Guttenburg contact.

Prominent joints were mapped in the excavation for the power plant block. The results of this excavation mapping program are presented in FSAR Attachment 2D. Joint patterns and characteristics identified in the excavation are similar to those found elsewhere on the site and in the site vicinity.

Analyses of the results of the seismic refraction survey indicate that the compressional wave velocities vary horizontally. Numerous low velocity anomalies have been interpreted along the five reconnaissance lines (Figures 2.5-65 to 2.5-73) and are shown on the profiles. These low velocity zones are thought to correspond to jointed, fractured bedrock which has undergone weathering and solution.

In the plant location, only one low velocity zone was interpreted along Line 8, Figure 2.5-72. Other joints have been inferred from air photo interpretation but as these joints did not appear as low velocity anomalies, they were thought to be relatively tight and unweathered.

More detailed discussions of solution activity along joints are presented in Subsection 2.5.1.2.6.

2.5.1.2.4.2 Folding

The regional structure at the site indicates a southeast dip of 0.25 degrees (approximately 15 to 25 feet per mile). Figures 2.5-27 and 2.5-28 show structure contours on the top of the Guttenburg Formation and the Harmony Hill Shale Member at the site and show good general agreement with the regional trend. In the plant location detailed drilling shows that minor variations do exist on these surfaces.

Figure 2.5-29 which shows structure contours on the Harmony Hill Shale Member in the plant location shows minor warping with the structural lineations being northwest-southeast and northeast-southwest. These lineations are in accord with the major regional structural trends. Figure 2.5-30, which shows structure contours on the Guttenburg Formation in the plant location, indicates general agreement with structure contours on the Harmony Hill Shale Member except the structural relief has been subdued.

2.5.1.2.4.3 Faulting

The faults have been divided into categories of major and minor faults. Major faults have a reported surface trace of over 1000 feet and minor faults a reported surface trace of less than 1000 feet. A tabulation of major reported faults in the midcontinent is given in Table 2.5-2 and shown in Figure 2.5-13. Table 2.5-3 lists all minor faults reported within a 12-mile radius of the site and Figure 2.5-16 shows all reported faults within 12 miles of the plant site, except those exposed in the excavation for the main plant structures, which are shown in FSAR Attachment 2.5C.

During the routine geologic investigation of the excavation at the Byron station, small vertical displacement faults (6 inches or less) were revealed. Preliminary notification of the discovery was given to the NRC on July 30, 1975. Geologists representing Sargent & Lundy, Dames & Moore, the Illinois State Geological Survey, the NRC, and the United States Geological Survey, visited the site and inspected the faults on August 6 and 7, 1975. A letter dated August 11, 1975 from Commonwealth Edison to the Directorate of Licensing described the faults and proposed a geotechnical investigation program designed to determine if the faults were "capable" according to the definitions of 10 CFR 100. A report entitled "Fault Specific Geotechnical Investigation" was transmitted to the NRC on August 29, 1975. Various photographs of test pits and stratigraphic cross-sections of these pits which were excavated along the trace of Fault 10-34 were transmitted to the NRC on September 16, 1975. Geologists representing Sargent & Lundy, Dames & Moore, the Illinois State Geological Survey, the NRC, and the United States Geological Survey, revisited the site and inspected the test pits on September 9, 1975 and were led on an inspection of the regional geology on September 10, 1975 by the Illinois State Geological Survey. On October 20, 1975 Commonwealth Edison provided additional supplemental responses to various questions raised by the NRC concerning the fault specific report.

The investigations of the faults at the Byron site by Sargent & Lundy and Dames & Moore and independent investigations by the Illinois State Geological Survey demonstrated that the faults were not "capable." Pertinent correspondence and reports on this subject are contained in FSAR Attachment 2.5C.

2.5.1.2.5 Groundwater

Groundwater conditions at the site are discussed in Subsection 2.4.13.

2.5.1.2.6 Solution Activity

Scattered small sinkholes are present within the immediate vicinity of the site (Figure 2.5-22 and Figure 2.5-101). These sinkholes are developed in the Dunleith Formation and have a maximum diameter of 150 feet (Figure 2.5-22) and appear to be located near the intersection of major joints. The Pleistocene deposits covering the site may be suppressing the surface expression of other small sinkholes. Outside the site area scattered sinkholes are present.

The exposed joints are narrow, open, vertical planar features. No "cave-like" openings have been observed. Joints exposed in the excavation for the main plant structures are described in FSAR Attachments 2.5C and 2.5D.

Southwest of the plant site area along Spring Creek there are several small springs. Ganymede Spring along the Rock River is the largest spring in the area with an estimated flow of 50 gpm (Peale, Reference 41). These springs issue from just above the Platteville-Ancell contact indicating that groundwater is percolating downward to the top of the Harmony Hill Shale Member where further downward movement is retarded. Residents of the area report other small intermittent springs which appear to issue from this zone or along the overburden-bedrock contact. Most of these springs are presently not flowing.

The Birdwell Division of Seismograph Services ran geophysical logs in Borings P-2, P-3, P-7, P-8, P-9, P-10, P-11, G-6, G-12, G-19, G-21, and G-22. These logs are presented in Figures 2.5-228 through 2.5-245. The interpretation of the anomalous zones shown on the geophysical logs are presented in the following paragraphs. The interpretations are based on the geophysical logs and visual examination of the rock core in each zone. Solution activity and void areas are not demonstrated on the graphic section of the logs but are described under the description section of the log.

The caliper log in Boring G-6 indicates a solution opening occurs at the Grand Detour-Mifflin contact zone between elevations 691 and 702 feet. The caliper log for Boring 12 shows that many thin irregularities, probably along bedding planes, occur in the Dunleith Formation mainly from elevations 796 to 799 feet and in the Grand Detour Formation from elevations 732 to 734 feet. In Boring G-19 many small irregularities are present; there is a small void at elevation 753 feet in the Nachusa Formation. In Boring G-22 the caliper log indicates a small opening in the Dunleith Formation from elevation 818 to 822 feet.

The suite of logs run at Boring G-21, which is suspected to lie near or on the intersection of two major joints, indicates the rock in the formations is highly fractured and solutioned from the top of the hole to an elevation of 677 feet in the Pecatonica

Formation. This is supported by data from the cores showing that the RQD did not exceed 28% and that core-recovery values did not exceed 62%. A severely fractured zone is present from elevation 677 to 695 feet along the Mifflin-Pecatonica contact zone in Boring G-21.

The suite of logs run in Borings P-2, P-3, and P-7, which are in the plant site directly under the reactor containment for Units 1 and 2 and in the center of the auxiliary building, indicate some minor irregularities and fracturing but generally indicate competent rock with little evidence of solution activity.

The suite of logs run in the other holes in the plant location all indicate some degree of fracturing and solution activity. In P-8, this is mainly apparent at the Nachusa-Grand Detour contact; in P-10, mainly in the lower Grand Detour and Miffin Formations; and in P-11, in the Nachusa Formation. In Boring P-9, the whole section from the Nachusa to the Pecatonica Formation shows marked fracturing and solution activity. The geophysical logs confirm the results obtained from analysis of RQD and core recovery values.

Examination of all borehole geophysical logs and cores found examples of both open and clay/sand filled fracture and solution zones in all formations down to the top of the Glenwood Formation.

Examination of outcrops and cores indicate that fracturing and weathering appears to decrease below the Dunleith-Guttenburg formational contact. This is evident from the fact that RQD values for the Dunleith are always low whereas in the formations below the Dunleith, RQD values are higher except near areas of major joints. Zones of voids, low core recovery, and low RQD have undoubtedly served as channels for the movement of groundwater, and examination of the cores suggests that solution activity has occurred along these channels.

The geophysical low velocity anomalies appear to be located along the trends of joint patterns. These may be weathered zones, solution features, or fracture zones through which groundwater infiltrates.

No evidence of solution activity has been found below the Platteville-Ancell contact. However, lower carbonate units and in particular the Potosi Formation may have cavities. The temperature log for Boring G-21 (Figure 2.5-230) shows two distinct groundwater temperature zones with the contact between these zones at the Platteville-Ancell contact. The temperature logs for Borings P-2, P-3, and P-7 (Figures 2.5-232, 2.5-234, and 2.5-236) were allowed to stabilize prior to logging and these logs also indicate a temperature change between these zones.

The chemical composition of the water from these two zones (Table 2.5-7) indicates that the water in the Galena-Platteville aquifer is higher in nitrates, sodium, calcium, aluminum, iron, lead, and lower in manganese than the water in the Cambrian-Ordovician aquifer. The Galena-Platteville aquifer also has a higher (more basic) pH than the Cambrian-Ordovician Aquifer.

From examination of the springs, cores, geophysical logs, outcrops, aerial photographs, piezometer data, and water analyses it seems that the lower limit of solution activity is stratigraphically controlled by the presence of the Harmony Hill Shale Member of the Glenwood Formation. It, therefore, appears that the Galena-Platteville aquifer in this area is not interconnected with the Cambrian-Ordovician Aquifer.

Solution activity is still going on at the site, but at a lesser rate than that occurring prior to the Wisconsin Stage. The majority of the solution activity is believed to have occurred before this period. From the beginning of the Wisconsin Stage and continuing to the present, the downward percolation of water has been retarded by the glacial deposits. Grouting of the main plant area down to the Platteville-Ancell contact will significantly retard the downward percolation of groundwater, and hence also limit the rate of solution activity. Results of the grouting program are included in FSAR Attachments 2.5A and 2.5B. Groundwater monitoring began on a monthly basis in December 1975. Chemical analyses are being performed on water samples from domestic wells to see if there is any change in the groundwater chemistry that would indicate an increase in the rate of solutioning of the bedrock. Results of the chemical analyses to date indicate no changes in the groundwater chemistry that may be attributed to an increase in the rate of solutioning (Subsection 2.4.13.4). Considering that the Byron site is located on a potentiometric high and that groundwater flow is approximately radially outward, it is not anticipated that there will be any increase in solution activity due to grouting for foundation improvement.

2.5.1.2.7 Man's Activities

There are no known instances of, or potential possibilities for, surface or subsurface subsidence, uplift, or collapse resulting from the activities of man within the site area. Present and former activities within this area have been surface quarrying of dolomite, removal of sand and gravel, and the domestic use of groundwater. There are no large uses of groundwater nor any industrial disposal wells in this area. No surface subsidence or response due to groundwater withdrawals has been reported near the site.

As shown on Figure 2.5-31, there are several quarries within 5 miles of the site. Only six of these quarries are presently active. There is also one sand and gravel operation near Byron. As these operations are all confined to the surface, no hazard is posed to the plant site because of surface subsidence.

The nearest active gas storage field, the Northern Illinois Gas Company's Troy Grove Project, is approximately 40 miles south of the site. During 1964 and 1965, the Natural Gas Pipeline Company of America tested the Mt. Simon Sandstone at the Brookville Dome, approximately 18 miles from the site, as a natural gas storage reservoir (Reference 13). The gas injected into the Mt. Simon Sandstone in this test passed upward through the Eau Claire Formation and Galesville Sandstone but was stopped in the Ironton Sandstone, immediately below the Franconia Formation. Further development of the site was not pursued since natural gas storage in the Franconia is not economical. After abandonment of the project by the Natural Gas Pipeline Company of America, gas was vented to the atmosphere through wells drilled to the Ironton and Galesville Sandstones. The venting of the gas took place from 1966 to 1974, when the wells were shut in. There have been no instances of up-life, subsidence, or collapse associated with these gas storage fields; therefore, no hazard is posed to the plant site because of these gas storage projects.

2.5.2 Vibratory Ground Motion

This section presents a discussion and evaluation of the seismic and tectonic characteristics of the Byron Station site and the surrounding region.

2.5.2.1 Seismicity

2.5.2.1.1 Seismicity Within 200 Miles of the Site

The North Central United States is among one of the areas of least seismic activity in the United States. Since this area has been populated for almost 200 years, it is likely that all earthquake events of Intensity VI or greater on the Modified Mercalli (MM) Scale (Table 2.5-8) which have occurred during this time span have been reported. Table 2.5-9 is a list of all known reported events which have occurred between 38° to 46° north latitude and 84° to 94° west longitude. The locations of these events and their spatial relationship to the area within a 200-mile radius of the site are shown on Figure 2.5-32. Within 200 miles of the site, 106 earthquakes have been known to occur. The largest events were three Modified Mercalli Intensity (MMI) VIIs which occurred in 1909.

The locations of the events listed in Table 2.5-9 and shown on Figure 2.5-32 which were located instrumentally are probably accurate to about ±0.1 degree. The location of older events, not determined instrumentally, many have occurred as much as ±1/2 degree from the stated location as the reported epicentral locations for these events normally correspond to the locations of the nearest reporting population center.

There is no record of any event larger than MMI VII occurring within 200 miles of the site. If such an event had occurred, it is almost certain that it would have either been recorded in private journals or diaries, or preserved in Indian legends as has been the case for other regions. The lack of such documentation indicates the absence of significant earthquake activity for a long period of time.

The most important earthquakes occurring within 100 miles of the site are:

a. 1804, Fort Dearborn, Illinois MMI VI - VII;

b. 1909, Beloit, Wisconsin, MMI VII;

c. 1912, northeastern Illinois, MMI VI;

d. 1934, Rock Island, Illinois, MMI VI; and

e. 1972, northern Illinois, MMI VI.

Isoseismal maps of the above earthquakes have been constructed for all but the 1804 Fort Dearborn event. Those for the 1909 and 1912 events which occurred approximately 55 and 25 miles from the site, respectively were prepared by J.A. Udden (References 42 and 43) and A.D. Udden (Reference 44) based on the Rossi-Forel scale of intensities which was in use at the time. These maps are reproduced here on Figure 2.5-33. The conversion to the Modified Mercalli Scale can be made using Table 2.5-8. The 1934 event (Figure 2.5-34) was felt no more than 50 miles from the epicenter (Reference 45). This puts the limit of perceptibility approximately 50 miles for the site.

Little is known about the Fort Dearborn earthquake of 1804 beyond a report of "quite a strong shock" (Reference 46) because most of Chicago's early records were destroyed in the Great Fire.

The 1972 northern Illinois earthquake had an Intensity of VI with its epicenter 35 miles south of the site (Figure 2.5-35). The shock was widely felt but did little damage (Reference 47).

One other significant event occurred within 200 miles of the site: the July 18, 1909 central Illinois event. This event was felt over a 35,000 mi^2 area and was probably felt at the Byron site (Reference 46).

2.5.2.1.2 <u>Distant Events</u>

2.5.2.1.2.1 <u>Central Stable Region</u>

Within the Central Stable Region only two other events were recorded which may have been felt at the Byron site. These are the 1909 Intensity VII southern Illinois event which occurred near Robinson, Illinois, approximately 225 miles from the site, and the 1968 Intensity VII southern Illinois event (Figure 2.5-36) which occurred near Broughton, Illinois approximately 290 miles from the site (References 46 and 48). Both these events occurred within the Wabash Valley area. This area is noted for a relatively high frequency of events, the largest of which has been Intensity VII (Reference 49).

2.5.2.1.2.2 Mississippi Embayment Area

The largest recorded earthquakes which have occurred in the central part of the United States were the New Madrid events of 1811-1812. These events occurred in the Mississippi Embayment area of the Gulf Coastal Tectonic Province (References 21, 49, and 50) at a distance of over 380 miles from the site (Figure 2.5-37 and Table 2.5-10).

Over a period of 3 months during 1811-1812, three large separate shocks occurred, the largest of which had an Intensity of XI-XII as well as at least 250 minor events (References 51 and 52). There has been no recurrence of such a major earthquake in this zone, but there is evidence of activity prior to the New Madrid events. There is a report of a very large shock on December 25, 1699 with its epicenter in western Tennessee, which shook approximately the same area as the 1811-1812 events. Written records also indicate that "notably vigorous" shocks occurred in 1776, 1791, or 1792, 1795, and 1804. Indian traditions also record a previous earthquake which devastated the same area (Reference 51).

In addition to these events, an Intensity VIII event occurred in 1895 in Charleston, Missouri also within the Mississippi Embayment area, which was probably felt at the Byron site.

2.5.2.1.2.3 Other Events

Two other events may have been felt at the Byron Station site: the 1886 Intensity X Charleston, South Carolina event which occurred in the Atlantic Coastal Province, and the 1935 Intensity VI Timiskaming, Canada event which occurred on the Canadian Shield. Details of these and other distant events are presented in Table 2.5-10.

2.5.2.2 Geologic Structures and Tectonic Activity

The Byron site and the entire 200-miles radius site region lie within the Central Stable Region of the North American Continent (Reference 21). This region is characterized by a relatively thin veneer of sedimentary rocks overlying a crystalline basement. These areas were deformed principally by movements which occurred as a result of tectonic activity culminating in the late Paleozoic into a series of gentle basins, domes, and other structures. Since the end of the Paleozoic, the area has remained generally quiescent.

The site is located on the flank of the Illinois Basin near the crest of the Wisconsin Arch. The most significant nearby structures are the Sandwich Fault Zone, the Plum River Fault

Zone, and the LaSalle Anticlinal Belt. A description of these and other tectonic features in the area is presented in Subsection 2.5.1.1.4.

2.5.2.3 Correlation of Earthquake Activity with Geologic Structures or Tectonic Provinces

The Central Stable Region Tectonic Province is generally noted for its lack of significant seismic activity. To evaluate the earthquake potential of the Byron site, two different approaches were utilized to correlate earthquake activity with geologic structures and/or tectonic provinces. By the first approach, the 200-mile radius site region was subdivided into seismotectonic regions utilizing methods similar to Stearns and Wilson (Reference 53). In the second approach, the site and its relationship to the Central Stable Region Tectonic Province and the Gulf Coastal Plain Tectonic Province was assessed, along with the relationship of the seismogenic structures of these provinces with the site.

2.5.2.3.1 Seismogenic Regions

Within 200 miles of the Byron Station, five seismogenic regions can be delineated, primarily on the basis of structure. These subdivisions are also indicative of the differing geologic histories of the seismogenic regions and of their varying seismic histories.

The following is a description of the five seismogenic regions within the 200-mile radius site area and other regions pertinent to the site. Each region is outlined on Figure 2.5-32.

2.5.2.3.1.1 Illinois Basin Seismogenic Region

The site is located on the north flank of the Illinois Basin Seismogenic Region. The north and northeastern boundaries of this region correspond to and are defined by the limits of the Plum River and Sandwich Fault zones.

The Byron Station site lies just south of the extension of the Plum River Fault Zone (Reference 7) and north of the Sandwich Fault Zone, and therefore, just within the Illinois Basin Seismogenic Region.

This region has experienced 60 recorded earthquakes, the largest of which were Intensity VI and Intensity VII. Some tentative correlation of events has been proposed by various authors, notably McGinnis and Ervin (Reference 54), who have postulated a correlation of earthquake events with areas of steep gradients in the earth's gravitational field which they interpret to indicate boundaries of crustal blocks. However, based on the present state of knowledge, these events are considered random. Therefore, the possibility of an Intensity VII event anywhere in the basin must be considered.

2.5.2.3.1.2 Ste. Genevieve Region

The Ste. Genevieve Region lies approximately 275 miles southwest of the site and is related to and defined by the imbricated Ste. Genevieve Fault Zone. This region exhibits a characteristic maximum intensity earthquake of MMI VI. While there is no evidence that this region and the included Ste. Genevieve Fault Zone are capable, fault plane solutions coincide with the trace of the fault (References 55 and 42). The boundary with the Illinois Basin is based both on a change in structure and by a contrast in seismicity.

2.5.2.3.1.3 Chester-Dupo Region

The Chester-Dupo Region, proposed by Nuttli (Reference 45), is defined by an area of faulting and folding in the vicinity of St. Louis. This region, approximately 210 miles southwest of the site, is one of moderate seismicity with maximum events characteristics of MMI VI-VII. The boundary between this region and the Illinois Basin is marked by the transition from the folds and faults of this region to the deeper, structurally less complex Illinois Basin. This region marks a hinge line between the Illinois Basin and the front elements of the Ozark Uplift.

2.5.2.3.1.4 Wabash Valley Seismogenic Region

This seismogenic region is defined by the limits of the Fairfield Basin, the deepest part of the Illinois Basin, and by the northwest-trending faults of the Wabash Valley. The closest approach of this region to the site is approximately 235 miles. This area has moderate seismicity with maximum events of the MMI VII. Events in this region occur more frequently than events in the adjoining parts of the Illinois Basin (Reference 49). The boundaries of the Wabash Valley Seismogenic Region can be well defined by structure and geological history as well as by its seismic pattern.

2.5.2.3.1.5 Iowa-Minnesota Stable Region

This region is one of extremely low seismicity with a general maximum intensity of MMI V. The boundary between this region and the Illinois Basin is approximately 65 miles from the site and is marked by a gentle zone of flexure, the Mississippi River Arch.

2.5.2.3.1.6 Missouri Random Region

The Missouri Random Region is bounded by the Chester-Dupo Region to the east, and its contact with the Illinois Basin Region is marked by the Lincoln Fold. This region lies approximately 185 miles southwest of the site. This area is characterized by the occurrence of random seismic events of maximum MMI V which are not associated with any known structure.

2.5.2.3.1.7 Michigan Basin Region

The Michigan Basin Region is an area of extremely low seismicity with a total of 10 recorded events. The largest was an MMI VI. This area is separated from the Illinois Basin by the Kankakee Arch and lies approximately 95 miles northeast of the site.

2.5.2.3.1.8 Eastern Interior Arch System Seismogenic Region

This region is composed of a series of gentle Paleozoic arches and domes within the eastern part of the Central Stable Region. Structurally, this area is composed of the Wisconsin, Kankakee, Findlay, and Cincinnati Arches, and the Wisconsin and Jessamine Domes. While this system can be subdivided into the various structures, the geological history of the structures and lithologies as well as general patterns of seismicity are similar. Since the boundaries between any of the structures are rather nebulous, divisions would be rather arbitrary.

The Wisconsin Dome in the northern part of the Central Stable Region consists of Precambrian rocks and is, therefore, more reflective of the Laurentian Shield subdivision of the Central Stable Region than the Interior Lowlands, the subdivision within the United States (References 21 and 50). The Wisconsin Dome is an extremely stable part of the Central Stable Region and represents the most seismically stable part of this region with maximum seismic activity of MMI V.

The Wisconsin Arch is defined structurally by the low, north-south trending, uplifted area extending south from the Wisconsin Dome and is herein defined as including the east-west trending crosscutting folds and faults of southern Wisconsin.

The boundary between the Wisconsin Arch and the Kankakee Arch is extremely hard to define. The name changes from the Wisconsin Arch to the Kankakee Arch northeast of Kankakee, Illinois. The Wisconsin Dome and Arch have a Precambrian core and are believed to have acquired their relief primarily by uplift whereas the relief on the Kankakee Arch is due primarily to more rapid subsidence of the bordering basins.

The arch system continues southeastward to join the Cincinnati Arch and the Jessamine Dome. The Findlay Arch is a northeastward splay off the Cincinnati Arch and separates the Michigan Basin from the Appalachian Basin.

Seismicity within this region is generally of MMI V. However, isolated events of MMI VII have occurred which cannot be related to specific structures. Therefore, the entire region must be assigned a maximum potential random event of MMI VII.

2.5.2.3.1.9 Anna Seismogenic Region

The Anna Region is at the intersection of the Kankakee, Findlay, and Cincinnati Arches in western Ohio. This area has experienced continued and moderately severe seismic activity. The largest historic earthquakes commonly have been of Intensity VII, with a single event of a maximum Intensity VII-VIII. This region is defined as lying within a basement structural zone bounded on the south by a northwest-trending band of basement faulting, on the east by a zone of structural weakness marked by a north-south trending band of magnetic highs and lows, on the north by a change from igneous extrusive to igneous intrusive rock, and on the west by the change from acidic extrusive to basic extrusive rocks. The combination of geological features within this area is unique. There is no other area within the central United States with the combination of factors similar to this region. The earthquake events which have occurred in this region are not random but rather the result of the unique combination of geological phenomena (Reference 56).

2.5.2.3.1.10 New Madrid Seismogenic Region

One of the most important seismogenic zones for determining maximum possible ground motion within the central United States is the New Madrid Seismogenic Region. This zone can be defined approximately on any tectonic map as corresponding to the northern portion of the Mississippi Embayment which is the northern portion of the Gulf Coastal Plain Tectonic Province (Reference 21, 49, and 50 and Figures 2.5-32 and 2.5-38).

The New Madrid events of 1811-1812 were the largest earthquakes ever experienced in the central and eastern United States. Chimneys were knocked down as far north as St. Louis, Missouri and the aftershocks from these events continued for 2 years (Reference 57). These events occurred more than 380 miles from the Byron site. Extensive studies have been conducted to determine the northern-most region in which these events could occur.

This has been documented in the Sargent & Lundy and Dames & Moore report, dated May 23, 1975, entitled, "Supplemental Discussion Concerning the Limit of the Northern Extent of Large Intensity Earthquakes Similar to the New Madrid Events" (Reference 58). Further discussion on this matter took place at a meeting held on January 26, 1976 in the offices of the Illinois State Geological Survey, Urbana, Illinois at the request of Public Service of Indiana. Representatives were present from the Nuclear Regulatory Commission, the Illinois State Geological Survey, the Indiana Geological Survey, the Kentucky Geological Survey, St. Louis University, Sargent & Lundy, Dames & Moore, and Seismograph Service Corporation (Birdwell Division). The scientific data presented clearly indicated that the New Madrid area, at the intersection of the Pascola Arch and the Ozark Dome, is tectonically unique and that the northernmost extent of the structurally complex New Madrid area is conservatively taken as 37.3° north, 89.2° west - 330 miles from the site. It remains the Applicants' position that, based on tectonic, geophysical, and seismic data, the New Madrid-type events should not extend across tectonic province boundaries and up the Wabash Valley Fault System.

More recent geophysical and seismological data also support the Applicant's position. Interpretations of gravity and magnetic data in Illinois (References 59 and 60), and others support the view that the Rough Creek Fault Zone separates distinct crustal provinces.

A regional microearthquake network has recently been installed in this area. Analysis of data obtained from this network indicates that the New Madrid region and the Wabash Valley Region are two distinct seismic regimes (Reference 61).

2.5.2.3.2 Tectonic Provinces

2.5.2.3.2.1 Central Stable Region Tectonic Province

The Central Stable Region is noted for its general lack of significant seismic activity with the largest events generally of MMI VII.

Within this tectonic province there are several zones of relatively high activity. These are (1) near Attica, New York, (2) near Anna, Ohio (3) the Wabash River Valley of southern Illinois and Indiana, (4) in Kansas and Nebraska along the midcontinent gravity and magnetic high in the area of the Nemaha Anticline and (5) near St. Louis, Missouri (Figure 2.5-38).

The Attica events are associated with the Clarindon-Lindon Structure and the August 12, 1929 events have been assigned an Intensity of VIII by Coffman and von Hake. However, the amount of damage and estimated magnitude of this event indicate that it was probably Intensity VII-VIII and that the assigned Intensity of VIII is extremely conservative (Reference 62).

The area around Anna, Ohio has experienced a relatively large amount of seismic activity compared to other areas of the Central Stable Region. As described previously, the area of earthquake activity corresponds to a highly complex Precambrian structural zone. In addition, the March 8, 1937 event, which has been assigned an Intensity VII-VIII by Coffman and Von Hake (Reference 46), has been analyzed and all indications are that this event had a maximum epicentral Intensity of VII (Reference 56).

The Wabash Valley Fault Zone was described in Subsection 2.5.2.3.1.4 and has had maximum recorded seismic activity of Intensity VII.

The midcontinent gravity and magnetic high in the area of the Nemaha Anticline has had several events of Intensity VII and the relationship of earthquake activity to the midcontinent gravity and magnetic high has been documented in Subsection 2.5.2 of the Wolf Creek PSAR (Reference 63).

The activity near St. Louis, Missouri has been assigned to the Chester-Dupo Region as defined by Nuttli (Reference 64) and documented in the PSAR for the Callaway Plant (Reference 65). Historical activity in this area has had a maximum Intensity VI-VII.

In addition to these areas of the Central Stable Region which have had relatively high seismic activity, an Intensity VIII event was reported in the Keewenaw Peninsula of Michigan in 1906 (Reference 46). The area of the epicenter is highly faulted, and the area of damage and perceptibility correspond to areas of mining activity. Smaller events which occurred earlier in the year as well as the larger event of 1906 all appear directly attributable to mining activity (Reference 66). The felt area of the 1906 event was approximately equal to that for an average Intensity III-IV event (Reference 46).

2.5.2.3.2.2 Gulf Coastal Plain Tectonic Province

The New Madrid events of 1811-1812 did not occur in the Central Stable Region Tectonic Province but in the Gulf Coastal Plain Tectonic Province. These events are associated with a highly complex structural zone near the crest of the Pascola Arch (see Subsection 2.5.2.3.1.10).

If these events are translated to the closest approach of this tectonic province to the site, these events could be expected to occur no closer than 330 miles from the site or 40 miles closer to the site than the 1811-1812 events occurred.

2.5.2.3.3 Earthquake Events Significant to the Site

By both methods of analyzing the tectonic association of earthquake events with structure, as described previously, the most significant earthquakes in the region are the 1909 Intensity VII Beloit earthquake, the 1972 Intensity VI northern Illinois earthquake, the 1912 Intensity VI northeastern Illinois earthquake, the 1804 Fort Dearborn earthquake, and the New Madrid earthquakes of 1811-1812. This evaluation is based on epicentral intensity, felt area, distance from the site and tectonic association.

2.5.2.4 Maximum Earthquake Potential

Based on the discussion in Subsection 2.5.2.3, the maximum earthquake which could be expected would be an Intensity VII event equivalent to the occurrence of an event similar to the 1909 Beloit Intensity VII event near the site. This is equivalent also to the occurrence of the largest event which has ever been recorded within the Central Stable Region, and which cannot yet be associated with a specific structure or structural region; it is therefore described as random. The level of ground motion experienced from a near field VII event would envelope the motion expected from a recurrence of a New Madrid-type event at the closest approach of the Mississippi Embayment, a distance of 330 miles from the site.

2.5.2.5 Seismic Wave Transmission Characteristics of the Site

The engineering properties of the soils and bedrock units at the site were evaluated using field geophysical measurements and laboratory testing; the properties determined by laboratory testing are discussed in Subsection 2.5.4.2.2.

Geophysical investigations performed at the plant site are presented in Subsection 2.5.4.4. The velocity of compressional and surface wave propagation and other dynamic properties of the natural subsurface conditions were evaluated from these investigations and the data were used in analyzing the response of the materials to earthquake loading.

Dynamic moduli for the subsurface soil and rock at the site were calculated based on measured properties. The in situ field measurements were compared with laboratory tests on the same materials. These analyses are presented in Subsection 2.5.4.7.

Seismic wave velocities and densities for the deeper rock strata in the region have been measured by others (Reference 59). These data confirmed field measurements, and were used in studies of site dynamic behavior.

2.5.2.6 Safe Shutdown Earthquake

The recommended safe shutdown earthquake was defined as the occurrence of an Intensity VII event near the site. This near field event would produce maximum horizontal ground accelerations of 0.13g (Reference 67 and Figure 2.5-39).

However, at the time of the review of the construction permit application, the NRC considered the occurrence of an earthquake of Intensity MM VIII to be equally probable (a low order of probability) at any place in the Eastern Central Stable Region. The NRC also took the position that, based on the postulated occurrence of an Intensity MM VIII at the site, a safe shutdown earthquake of 0.2g at the bedrock-soil interface was adequately conservative for the Byron Station. This value was applied at the foundation level. Utilizing the subsurface properties presented in Subsection 2.5.4.7, the corresponding ground surface acceleration was found to be 0.21g. This would be the controlling seismic event even if a New Madrid-type event were postulated to occur at Vincennes, Indiana more than 235 miles from the site. These conclusions are based on information presented in Reference 56.

The ground response spectra prepared following the guidelines of Regulatory Guide 1.60 for a horizontal ground acceleration of 0.21g are presented on Figure 2.5-40.

2.5.2.7 Operating Basis Earthquake

The operating basis earthquake (OBE) is intended to indicate those levels of ground motion which could reasonably occur at the plant site.

On the basis of the seismic history of the area, it appears very unlikely that the site will be subjected to any ground motion of significant levels during the life of the nuclear power station.

It is probable that the maximum level of ground motion experienced at the site during historic time was Intensity VI and was due to the 1909, Intensity VII, Beloit earthquake. For this intensity, the maximum horizontal ground acceleration at the site was probably on the order of 0.06g. Therefore, the OBE acceleration at the bedrock surface was conservatively recommended to be 0.06g for horizontal ground motion.

In addition, a probability analysis (Reference 68) of the occurrence of earthquakes at the Braidwood Station was also performed using the data on past earthquakes in the area. The results of this analysis are equally applicable to the Byron Station. In performing this analysis, epicenters were assumed to occur at random in a 195,000 mi^2 area around the station. The results of this probability analysis show that a site Intensity of MMI VI has an average return period of 2150 years. Because of this long return period, the site Intensity of VI was selected conservatively as the OBE. However, the NRC took the position that an OBE of 0.09g at the bedrock-soil interface was conservative for the Byron site. The acceleration level for the OBE was therefore selected at 0.09g. It should be pointed out that this acceleration level is higher than the mean level of acceleration for an Intensity VI event and corresponds approximately to acceleration levels expected for a VI-VII event (Reference 67 and Figure 2.5-39). Additional conservatism was then used as the 0.09g acceleration level was applied at foundation levels utilizing the subsurface properties presented in Subsection 2.5.4.7. The resulting horizontal ground acceleration at the ground surface was 0.095g.

The response spectra for 0.095g horizontal ground acceleration prepared following the guidelines of Regulatory Guide 1.60 are presented as Figure 2.5-41.

2.5.3 Surface Faulting

There was no evidence found of surface faulting or of faulting of the soil materials overlying bedrock at the site or within 5 miles of the site (Subsection 2.5.1.2.4.3). The unfaulted soil materials at the site consist of Pleistocene-age glacial tills (the oldest of which is Illinoian or older in age) which overlie a residual soil that is Yarmouthian or older in age, and is most likely Tertiary in age (FSAR Attachment 2.5C). Several faults of small displacement were noted in the underlying Ordovician-age bedrock during excavation for the main plant. The characteristics, age, and origin of these faults are discussed in the Fault Specific Geotechnical Investigation presented in FSAR Attachment 2.5C.

In the regional area the nearest major faults (defined herein as faults with a surface trace of more than 1000 feet) are the Sandwich Fault Zone, which terminates approximately 7 miles south of the site and the Plum River Fault Zone, which terminates 5.3 miles west-northwest of the site (Subsection 2.5.1.1.4.2).

There are no known capable faults in the regional area (200-mile radius).

2.5.3.1 Geologic Conditions of the Site

A discussion of the lithologic, stratigraphic, and structural geologic conditions, the geologic history of the site, and the surrounding region is presented in Subsections 2.5.1.1 and 2.5.1.2.

2.5.3.2 Evidence of Fault Offset

There is no evidence of fault offset at the ground surface at the site. An investigation of the faults of small displacement found in the main plant excavation is presented in FSAR Attachment 2.5C.

Other minor faults with a surface trace of less than 1000 feet, and occurring within 12 miles of the site are discussed in Subsection 2.5.1.1.4.2 and Table 2.5-2, and are illustrated in Figure 2.5-16.

2.5.3.3 Earthquakes Associated with Capable Faults

There have been no historically reported earthquakes within 5 miles of the site. No capable faulting is known to exist within 5 miles of the site nor within the regional area.

2.5.3.4 Investigation of Capable Faults

No investigation was required since there are no known capable faults in the regional area.

2.5.3.5 Correlation of Epicenters with Capable Faults

No capable faulting is known to exist within 5 miles of the site, and no earthquake epicenter is known within 5 miles.

2.5.3.6 Description of Capable Faults

No capable faulting is known to exist within 5 miles of the site.

2.5.3.7 Zone Requiring Detailed Faulting Investigation

A detailed fault investigation was performed on site to determine the age and origin of faults in the bedrock which were noted in the main plant excavation. They were found to be noncapable because of their age (the last movement was at least pre-Yarmouthian) and because they may in fact be nontectonic in origin. The results of the Fault Specific Geotechnical Investigation are presented in FSAR Attachment 2.5C.

2.5.3.8 Results of Faulting Investigation

The results of the Fault Specific Geotechnical Investigation are presented in FSAR Attachment 2.5C.

2.5.4 Stability of Subsurface Materials and Foundations

This subsection presents an evaluation and summary of the geotechnical suitability and stability of the subsurface materials to support the plant foundations. A general site plot plan is shown on Figure 2.5-3.

2.5.4.1 Geologic Features

A detailed discussion of the geologic characteristics of the site is given in Subsection 2.5.1.2 and FSAR Attachment 2.5D. A comprehensive field and laboratory investigation program including borings, test pits, geophysical surveys, field reconnaissance, and various static and dynamic laboratory tests was undertaken to determine the geologic features at the site and their significance with relation to site suitability and stability.

2.5.4.2 Properties of Subsurface Materials

This subsection presents the static and dynamic engineering properties of the soil and rock at the site. The properties are based upon review and analysis of field and laboratory tests performed during this investigation.

2.5.4.2.1 Field Tests

A program of field investigations was undertaken to evaluate the materials underlying the station site. A detailed discussion of the results of these investigations is presented in Subsections 2.5.4.3 and 2.5.4.4.

2.5.4.2.2 Laboratory Tests

The program of laboratory tests undertaken to evaluate the materials underlying the station site consists of the following:

a. static tests

1. unconfined compression,
2. particle size analysis,
3. Atterberg limits,
4. moisture and density determinations, and
5. consolidation tests.

b. dynamic tests

1. cyclic triaxial compression,
2. shockscope, and
3. resonant column.

2.5.4.2.2.1 Static Tests

2.5.4.2.2.1.1 Unconfined Compression Tests

Unconfined compression tests were performed on representative soil and rock samples to determine their strength characteristics. The strength of the soil samples was determined from load-deflection curves obtained from unconfined compression test data. Determinations of the field moisture content and dry density of the soil were made in conjunction with each test. The results of the tests on soil and the corresponding moisture contents and dry-density determinations are presented on the boring logs (Figures 2.5-104 through 2.5-227).

The strength of the underlying rock formations was evaluated by subjecting representative rock core samples to unconfined compression tests. Tests were performed by the Robert W. Hunt Company, Chicago, Illinois, in accordance with the American Society for Testing and Materials D2938-71 standard testing procedures. Samples approximately 4 inches in height and 2 inches in diameter were subjected to a constant rate of axial load. The results of the rock compression tests are presented in Table 2.5-11.

Poisson's ratio and the modulus of elasticity were determined on two rock samples from the Dunleith Formation by attaching two strain gauges on the sides of the core diametrically opposite each other and parallel with load and two additional gauges opposite each other and perpendicular with load. The results of the average of the two readings was used to determine Poisson's ratio. The results of the Poisson's ratio determination and the modulus of elasticity are presented in Table 2.5-11.

2.5.4.2.2.1.2 Particle Size Analysis

Representative soil samples were analyzed to determine their particle size distribution. The test results are presented on Figures 2.5-42 through 2.5-51.

2.5.4.2.2.1.3 Atterberg Limits

Representative soil samples were tested to evaluate their plasticity characteristics. The results of these tests were used for classification and correlation purposes. The Atterberg limit determinations are presented on the boring logs (Figures 2.5-104 through 2.5-185 and Figures 2.5-204 through 2.5-226).

2.5.4.2.2.1.4 Moisture and Density Determinations

In addition to the moisture and density determinations made in conjunction with the unconfined compression tests, independent moisture, and density tests were performed on other undisturbed soil samples for correlation purposes. The results of all moisture and density determinations are presented on the boring logs (Figures 2.5-104 through 2.5-185, 2.5-204 through 2.5-277, 2.5-190, 2.5-191, 2.5-193, and 2.5-199). Data on material similar to the structure backfill using the Modified Proctor Test (ASTM-D1557-70) and values for maximum and minimum density obtained using the ASTM-D2049-69 procedure are given in Table 2.5-32. The gradations for the crushed rock material are shown in Figure 2.5-248. As may be seen, from Table 2.5-32, 95% of the maximum dry density as determined by modified Proctor corresponds to relative densities in excess of 95%. Therefore, the Modified Proctor Test is considered both appropriate and conservative.

2.5.4.2.2.1.5 Consolidation Tests

Consolidation tests were performed on representative soil samples to determine the compressibility characteristics of the soils. The method of performing consolidation tests is described on Figure 2.5-52. The results of the consolidation tests are presented in Figures 2.5-53 through 2.5-56.

2.5.4.2.2.2 Dynamic Tests

A detailed discussion of dynamic tests is presented in Subsection 2.5.4.7.2.

2.5.4.3 Exploration

2.5.4.3.1 General

A series of explorations were conducted to identify the geologic, groundwater, and foundation characteristics of the site. These investigations include soil and rock borings, installation of piezometers, water pressure test, test pits, surface seismic, and borehole geophysical surveys.

The limits of the plant excavation are shown on Figures 2.5-57 and 2.5-58. Geologic sections through Seismic Category I structures are presented on Figures 2.5-59 and 2.5-60.

The limits of the river screen house excavation are shown on Figure 2.5-246. The location of the essential service water makeup pipeline, a cross section through the essential service water makeup pipeline and circulating water pipeline, profiles along the essential service water makeup pipeline, and generalized geologic sections along the essential service water makeup pipeline are presented in Plates 1, 2a, 3a through 3g, and 4a through 4c, respectively, in FSAR Attachment 2.5G.

2.5.4.3.2 Test Borings

2.5.4.3.2.1 Site Geologic Test Borings

Twenty-five widely spaced geologic borings (G-Series) were drilled at the site from August through October 1972, by Raymond International, Inc., under the supervision of Dames & Moore. The purpose of the borings was to determine the details of the lithology, structure, and physical properties of the subsurface strata. The borings range in depth from 98 to 317 feet below the ground surface and were drilled at the locations indicated on Figures 2.5-3 and 2.5-5.

The drilling was done with truck-mounted rotary wash equipment. The borings were generally drilled without the use of driller's mud; however, revert mud was utilized in the lower portions of the borings. Revert mud has the consistency of other drilling muds when mixed, but since it is biodegradable, it reverts back to the consistency of water with time and therefore does not impair subsequent water-pressure tests.

Detailed descriptions of the soil and rock encountered in the borings are presented in Figures 2.5-104 through 2.5-140. The soils were classified according to the Unified Soil Classification System described in Figure 2.5-103.

Rock was cored utilizing NX double-tube core barrels, which provide rock cores approximately 2 inches in diameter. Undisturbed soil samples suitable for laboratory testing were obtained using a Dames & Moore Type U Sampler. The sampler is 3-1/4 inches in outside diameter and approximately 2-1/2 inches in inside diameter. Disturbed samples were extracted utilizing a standard split-spoon sampler approximately 2 inches in outside diameter and 1-3/8 inches in inside diameter. These samples were taken using the Standard Penetration Test procedure.

The borings were water-pressure-tested using inflatable packers and a 10-foot packer spacing with the entire interval between the packers consisting of perforated pipe. In the deeper portion of the borings (approximately 50 feet below the surface) three tests were performed at each interval; for shallower depths, two tests were performed. A minimum gauge pressure of 20 psi and a maximum of 250 psi were used with the inflatable-type packers. Maximum net water-pressure used was 1.1 psi per foot of depth. With every interval tested, the low pressure was repeated as the final test for the interval. The purpose for this was to see if the higher pressure had washed any fines from the joint fillings, thereby increasing the water take of a zone. The results of the pressure tests, are presented on the boring logs (Figures 2.5-104 through 2.5-140). These results are presented as ranges for each interval tested as water loss units which are computed to the following formula:

$$\text{Water Loss Unit} = \frac{\text{Rate of loss gallons per minute}}{\text{Interval tested in feet x Net pressure in psi}}$$

The net pressure is given as:

Net pressure = Gauge pressure + Column pressure - Friction loss.

The column pressure is equal to the depth to the upper packer or the depth in feet to groundwater, whichever is smaller, times a constant of 0.433. The friction-loss was determined by a standard friction-loss test in the field. A computer program was drawn up to reduce the field data.

Nineteen auger borings (A-series) were drilled from August through October 1972 by Raymond International, Inc. under the supervision of Dames & Moore at the locations on Figure 2.5-3 to evaluate the depth to bedrock. The results are summarized in Table 2.5-12.

2.5.4.3.2.2 Plant Site Test Borings

Seventy-five borings (P and T-series) were drilled at the site by Raymond International Inc. under the supervision of Dames & Moore. The purpose of the borings was to obtain samples for the determination of the details of the lithology, structure, and physical properties of the subsurface strata. The borings range in depth from 9 to 266 feet below the ground surface and were drilled at the locations indicated on Figures 2.5-4 and 2.5-7. The drilling and sampling methods are the same as described in Subsection 2.5.4.3.2.1.

Detailed descriptions of the soil and rock encountered in the borings are presented in Figures 2.5-141 through 2.5-185 and 2.5-204 through 2.5-227. The soils were classified according to the unified soil classification system described in Figure 2.5-103.

Twelve of these borings were water pressure tested using the same method and equipment described in Subsection 2.5.4.3.2.1. The results are presented in the boring logs and Table 2.5-6.

2.5.4.3.2.3 River Screen House Test Borings

Seven borings (G-23 and RS series) were drilled at the location of the river screen house by Raymond International, Inc. under the supervision of Dames & Moore. The purpose of the borings was to obtain samples for the determination of the details of the lithology, structure, and physical properties of the subsurface strata at the river screen house. The borings ranged in depth from 111.5 to 135 feet below the ground surface at the locations shown on Figure 2.5-5. The drilling and sampling was done utilizing the same methods and procedure described in Subsection 2.5.4.3.2.1.

Detailed descriptions of the soil and rock encountered in the borings are presented on Figures 2.5-138 and 2.5-186 through 2.5-191.

The soils were classified according to the unified soil classification system described in Figure 2.5-103.

2.5.4.3.2.4 Essential Service Cooling Tower Makeup Pipeline Test Borings

Eighteen borings and thirty auger borings were drilled for the makeup pipeline investigation under the supervision of Dames & Moore. The purpose of the borings (I and IB series) was to determine the details of the lithology, structure, and physical properties of the subsurface strata along the makeup pipeline route. These borings ranged in depth from 5 to 115 feet below the ground surface at the locations shown on Figure 2.5-6. Three borings (SH series) were drilled at the locations shown on Figure 2.5-102 in December 1973 to evaluate the age of the 150-foot diameter sinkhole discussed in Subsection 2.5.4.10.4. Detailed descriptions of the soil and rock encountered in these borings are presented on Figures 2.5-102 and 2.5-192 through 2.5-203. The soils were classified according to the unified soil classification system described in Figure 2.5-103. A solution enlarged joint exposed during excavation of the essential service water makeup pipeline in 1977 at Station 52+75W of the corridor has been discussed in FSAR Attachment 2.5G. In this section of the pipeline excavation, grade was about 9 feet below the top of bedrock. The solution feature was along an east-west joint and varied from 1 to 1-1/2 feet in width and extended approximately 12 feet below grade. The solution feature was photographed, mapped, and filled with approximately 1-1/2 cubic yards of concrete. The concrete was well vibrated and placed in stages.

Thirty auger borings (IA and IAB series) were drilled under the supervision of Dames & Moore at the locations shown on Figure 2.5-6 to determine the depth to bedrock and auger refusal. The borings were drilled in 1977 to investigate the soil conditions in Area of Concern No. 11 and other sections of the pipeline corridor. Boring IB-3 (FSAR Figure 2.5G-6B) was cored into bedrock. Although small vugs (pinpoint to less than 0.5 inch diameter holes) were present in 1% to 5% of the rock core, no evidence of cavities was found. Furthermore, based on field reconnaissance and aerial photograph interpretation performed during the PSAR studies, no sinkholes were found to exist in this area.

The investigations presented in FSAR Attachment 2.5G were performed to evaluate the strength and liquefaction potential of the materials underlying the pipeline. This concern was resolved by the liquefaction analysis presented in FSAR Attachment 2.5G which concluded that the soils in Area of Concern No. 11 are not susceptible to liquefaction. Three test pits were excavated at the locations shown on Figure 2.5-6 to correlate excavation characteristics with auger refusal. The results of the auger borings and test pits are summarized on Table 2.5-13.

2.5.4.3.3 Piezometers

The locations, installation, and monitoring of piezometers are discussed in Subsection 2.4.13.2.3.

2.5.4.3.4 Geophysical Surveys

Surface seismic and borehole geophysical surveys conducted at the site are discussed in Subsection 2.5.4.4.

2.5.4.3.5 Excavation Mapping

Detailed mapping of the excavation for Seismic Category I structures was performed by Dames & Moore. The geologic subsurface conditions found during the excavation mapping program generally confirm the interpretation made from the subsurface investigation in the PSAR. Excavation mapping for the plant structures is presented in FSAR Attachment 2.5D. Surveillance reports for the river screen house and essential service water makeup pipeline excavations are provided in FSAR Attachments 2.5H and 2.5G, respectively.

2.5.4.4 Geophysical Surveys

The following site geophysical surveys were conducted:

a. a seismic refraction survey to evaluate compressional wave velocities,

b. a surface and shear wave survey to determine surface wave types and characteristics and shear wave velocities,

c. an uphole velocity survey to define compressional wave velocities,

d. a downhole shear wave survey to evaluate shear wave velocities,

e. ambient noise studies to determine the predominant frequencies of ground motion of the site due to background noise levels, and

f. geophysical borehole logging to further establish the geophysical characteristics and variations in each of the geologic units encountered.

A description of each phase of the geophysical explorations is provided in the following paragraph along with a typical geologic profile showing geophysical properties on Figure 2.5-62.

2.5.4.4.1 Seismic Refraction Survey

Seismic refraction surveys were conducted by Dames & Moore and Geoterrex, Ltd. to evaluate the subsurface characteristics of the site in general, and to confirm the nature of the underlying strata as established by cross-sections based on geologic borings. The survey was conducted at the site along nine seismic lines for a total length of 24,700 lineal feet. The seismic lines were located as shown on Figures 2.5-63 and 2.5-64.

Seismic energy was produced by the detonation of a small explosive charge in both drilled and hand-dug holes. The energy released by the explosives was picked up by vertically oriented geophones fitted with a spike for coupling with the underlying soil. Hall-Sears geophones, having a natural frequency of 4.5 Hz, were spaced at 50-foot intervals along the seismic refraction lines.

The seismic energy was recorded by a 24-channel Dresser S.I.E. RA-44 seismic amplifier coupled with a Dresser S.I.E. R-24A recording oscillograph and a 12-channel Electro-Tech Labs ER-72-12A seismograph.

The geophysical field crew consisted of two geophysicists, an operator, a licensed powderman, a helper, and a driller and helper. The field work was performed from August 27 to September 16, 1972, and from October 31 to November 4, 1972.

Compressional wave velocities and the depths to various subsurface layers under the site were evaluated by plotting the first arrival times of the seismic energy at each geophone location against the distance of each geophone from the source of the seismic energy. The time-distance data from each profile are shown on Figures 2.5-65 through 2.5-73. In addition, profiles of the various velocity zones are shown directly below each corresponding time-distance plot. In using the time-distance plots, note that they reflect information collected from shot points made at several locations along a seismic line. For clarification on the figures, two plot symbols have been used to indicate the origin of the geophysical shots: from the left (.), from the right (Δ). The subsurface section shown represents an evaluation of the most probable conditions, based upon interpretation of presently available data. Some variation from these conditions should be expected.

A seismic refraction reconnaissance survey was completed along the initial five refraction lines, Lines 1 through 5 on Figures 2.5-65 through 2.5-69. The purpose of the reconnaissance survey was to generally define the subsurface conditions along these lines throughout the plant site. The plant location was studied in detail by four additional seismic refraction lines, Lines 6 through 9 on Figures 2.5-70 through 2.5-73.

The seismic refraction reconnaissance survey detected an initial or surface unit with a velocity of approximately 1000 ft/sec, generally underlain by a bedrock velocity unit of approximately 7500 ft/sec. These layers are in turn underlain by an approximate 12,000 ft/sec unit and a 15,500 ft/sec unit. In the vicinity of the plant site these velocity units generally correspond with the underlying lithology with the interface between the 12,000 and 15,500 ft/sec velocity units corresponding to the approximate top of the Guttenburg Formation. The 7,500 and 12,000 ft/sec velocity units reflect differences in fracturing and weathering which the bedrock has undergone rather than lithologic differences. Several low velocity zones were detected within bedrock velocity units and these zones are thought to correspond to jointed, fractured bedrock which has undergone weathering and solution.

The low velocity zone at Station 30+50 in Figure 2.5-65 (Seismic Refraction Survey Line 1) is shown as an anomaly within the 12,000 and 15,000 feet per second (third and fourth) layers (depth of 60 to 70 feet) and does not evidence itself in the 7500 feet per second (second) layer. The bedrock surface represented by the interface between the 1000 and 7500 feet per second (first and second) layers shows no evidence of depression above this low velocity zone. For these reasons, this zone is not considered to represent the presence of a sinkhole. Therefore, the zone has no impact on pipeline design.

The low velocity zone at Station 37+00 in Figure 2.5-65 (Seismic Refraction Survey Line 1) is shown as an anomaly within the 12,000 and 15,500 feet per second (third and fourth) layers. This zone is overlain by the occurrence of 5000 feet per second material which occurs in place of the 7500 feet per second (second) layer. No drill holes penetrated the subsurface materials at this location, hence the indirect interpretation from the seismic refraction data is the only source of information on this feature. As indicated in Subsections 2.5.1.2.6 and 2.5.4.4.1, the low velocity zones appear to be located along the trends of joint patterns and represent weathered zones, solution features, or fracture zones. Because of the presence of the 5000 feet per second material in the second layer, it is possible that this zone could be attributed to a collapsed solution feature of Wisconsinan to pre-Pleistocene age similar to the 150 foot diameter sinkhole discussed. This is based on the fact that it does not have a topographic manifestation and could not be detected on air photos. If this zone represents a solution feature, the feature would have been formed during Wisconsinan to pre-Pleistocene time and does not represent a Holocene age feature.

The four seismic refraction lines completed within the plant site generally revealed higher seismic velocities than those detected on the reconnaissance lines which relate to the more competent bedrock encountered in the plant area. Generally in the plant site area a near surface unit having a velocity from 1,400 to 2,200 ft/sec and is underlain by a bedrock velocity unit ranging in velocity between 8,500 and 11,000 ft/sec. This unit is in turn underlain by a bedrock velocity unit ranging in velocity between 13,500 and 15,250 ft/sec. The interface between the two bedrock velocity units correlated with the approximate top of the Guttenburg Formation.

Seismic Line 6, (Figure 2.5-70), is located along the eastern edge of the structures, primarily underlying the heater bay. Generally high velocities were observed across the profile. Velocities in the upper bedrock velocity unit ranged between 10,000 ft/sec and velocities in the lower bedrock velocity unit ranged between 14,500 and 15,250 ft/sec. These high velocities correlate with the generally high RQD values.

Seismic Line 7 (Figure 2.5-71), which passed through the center of the reactor containment, shows generally high bedrock velocities and RQD values are generally high along this line. The velocity units show little horizontal variation with velocities in the upper bedrock velocity unit of approximately 10,000 ft/sec and in the lower velocity unit of approximately 15,000 ft/sec.

Seismic Line 8, (Figure 2.5-72), lies west of the plant structures and moderate velocity variation is noted in the upper bedrock velocity unit ranging from 9,500 ft/sec to 10,500 ft/sec. The lower bedrock velocity unit has a velocity of 14,000 ft/sec with the exception of the area between approximate station 3+00 to 6+00. Between those stations a low velocity zone was detected which ranged in velocity between 9,500 ft/sec and 11,000 ft/sec and may be a weathered or fracture zone related to a joint system. The lower bedrock velocity unit is noted as increasing in elevation near both ends of the profile.

Seismic Line 9, (Figure 2.5-73), is an east-west profile through the center of the plant site. Variable velocities were observed along this profile with velocity variation in the upper bedrock velocity unit ranging from approximately 8,500 ft/sec in the western portion to 11,000 ft/sec in the eastern portion. This corresponds with a general increase in RQD values in this unit in an eastward direction. The lower bedrock velocity unit ranged in velocity between 13,500 ft/sec to 15,000 ft/sec.

Knowledge of the local geologic conditions dictates the interpretation of highly weathered zones at the low velocity anomalies. The low velocity anomalies nearest the plant site have been investigated by Dames & Moore by drilling borings in these areas. The low velocity anomaly between station 3+00 to 6+00 on seismic line 8200 feet west of the plant, is caused by the highly weathered zone related to the intersection of two joints, as indicated by Boring P-21. The low velocity anomaly encountered at station 40+00 on seismic line 3, 1200 feet south of the plant, is caused by a highly weathered zone as indicated by Borings G-10 and G-11 and shown on Figure 2.5-74.

2.5.4.4.2 <u>Surface and Shear Wave Velocity Survey</u>

In order to further evaluate dynamic bedrock characteristics, a surface wave and shear wave velocity survey was conducted by Dames & Moore and Geoterrex, Ltd. along a 2300-foot section trending northeast-southwest (Figure 2.5-63).

Surface and shear wave velocities were computed by Dames & Moore and Geoterrex, Ltd. from the recordings of two, three-component, Sprengnether Engineering Seismograph seismometers in conjunction with a Dresser S.I.E. R-24A recording oscillograph. The Sprengnether seismometers were placed 350 feet apart with explosions detonated at varying distances, from 1300 to 2300 feet, from these seismometers.

The surface waves generated at this site by small explosions are relatively small in amplitude, apparently due to the high attenuation characteristic of the soil and bedrock. The characteristics of the surface waves observed in this study are given in Table 2.5-14.

Basically, one type of surface wave was recorded, the Sezawa M_2 branch of the Rayleigh family. It appears to be a highly dispersed train, having a maximum velocity of 3730 ft/sec and a frequency of 16 Hz. This wave is typical of the predominant wave developed when the source of energy is a small explosive charge detonated at a shallow depth. This wave is probably not the first mode of the site. Earthquake excitation of the site would probably produce the first mode; however, from the studies, the site would probably be relatively insensitive to the first mode.

The observed surface wave has a very high velocity, high frequency, and progressive elliptical motion with a radial to vertical amplitude ratio of more than 3 to 1. This wave would then be classified as a modified "sezawa" or M_2 branch of the Rayleigh surface wave family. These characteristics suggest that this wave is probably not a first mode excitation of the site. The lack of lower frequency surface waves even on the longest distance records (2300 feet) where first or lower mode surface waves might be expected, indicates that the site will probably be relatively insensitive to the first Rayleigh modes of the near surface family of surface waves, when excited by an earthquake. The site does not have material properties and a geometry capable of producing large-amplitude, quasi-resonant, near-surface waves under any excitation except for the M_2 type observed on the field records. With this exception, this site, therefore, does not possess any high-amplification (high Q factor) peaks within the site-response spectrum at frequencies above about 2 Hz, caused by quasi-resonances within the near-surface family of surface waves.

The information obtained from the field records is analyzed for compressional wave information, secondary arrival information, and for surface waves. This analysis is based on the geologic model of the site which is derived from both geophysical and geological information. This data is plotted as time versus distance for velocity determinations. In addition, a particle motion analysis is performed on each surface wave for a determination of surface wave type. A time-distance plot for the surface waves of this site is shown on Figure 2.5-75.

Secondary arrivals occurring between the compressional wave arrival and the surface waves are analyzed for motion, frequency content, cyclic change, and character and are used in the site model for confirmation of shear wave velocity, if in fact the arrivals are determined to be shear wave arrivals. The plot of two of these arrivals is also shown on Figure 2.5-75. The analysis of the secondary arrivals is difficult, as some of these arrivals are refracted-reflections and reflected-refractions of the compressional wave arrivals.

On one short refraction profile, a weathering spread with 10-foot geophone spacing, between Borings G-11 and G-12, a secondary arrival was picked behind the compressional wave arrival for the 7,500 to 11,000 ft/sec zone. When this arrival was corrected, it corresponded to a shear wave velocity of 2900 to 3500 ft/sec.

2.5.4.4.3 Uphole Velocity Survey

An uphole velocity survey was performed by Dames & Moore and Geoterrex, Ltd. in Boring G-12 to provide a check on the compressional wave velocities measured during the seismic refraction surveys. The boring was cased to 5 feet below the ground surface with plastic casing.

Small explosive charges were detonated at 10-foot intervals within the boring. The seismic response to the explosive charges was detected on the ground surface by three, Hall-Sears, 4-1/2 Hz geophones and recorded on an Electro-Tech Labs ER-72-12A seismograph. The results of the uphole velocity survey are presented on Figure 2.5-76.

It can be seen that the compressional wave velocities measured from this survey differ slightly from the compressional wave velocities measured in the seismic refraction surveys. The seismic refraction survey measures and averages the compressional wave velocities over a larger distance, whereas the uphole velocity survey measures the compressional wave velocities at an isolated point (Boring G-12).

2.5.4.4.4 Downhole Shear Wave Survey

The downhole shear wave survey was performed by Dames & Moore and Geoterrex, Ltd. utilizing Borings G-12 and G-13. A three-component, low frequency, borehole geophone (Mark Products LI-3DS) was lowered into each boring and maintained at various depths.

Explosive charges were fired at distances ranging from 500 to 1000 feet from Boring G-13. The resultant seismic energy was recorded on a Dresser S.I.E. RA-44 seismic amplifier coupled to a Dresser S.I.E. R-24A recording oscillograph. Recordings were made of the seismic energy of successive 50-foot intervals in Boring G-13. This procedure could only be followed in the lower portion of Boring G-13 as a coupling fluid could not be maintained above approximately 80 feet.

The information obtained from the downhole survey is used for the determination of shear wave velocity, and the confirmation of compressional wave velocity. For this site, the downhole, survey was, in fact, a standard crosshole survey.

By placing the shot points, and the two borings along one of the refraction profiles, the subsurface conditions are known for the purpose of building a model. The compressional wave velocities as determined from the refraction survey are used in this model. By knowing the compressional wave velocities, the depth, and the thicknesses of each subsurface layer for the model, ray path diagrams are constructed from each shot point to each of the downhole geophones for each shot. With this model, compressional wave arrival times can be calculated for each shot at each geophone. These calculated times are then checked with actual arrival times found on the field records. Once the model and the ray paths have been corrected to match compressional wave arrival times, the procedure is repeated for secondary arrivals. The secondary arrivals are also analyzed for frequency content, cyclic change, and character.

The data obtained from this survey is tabulated in Table 2.5-15.

2.5.4.4.5 Ambient Vibration Measurements

Measurements of the ambient background motion of the site and its response to natural motion generators are indicative of the site's dynamic properties.

These measurements were made by Dames & Moore and Geoterrex, Ltd. at three locations on the site, shown on Figure 2.5-63. The measurements were made during relatively quiet periods.

A three-component, direct-writing, Sprengnether Engineering Seismograph Model VS-122, was used for recording ambient ground motion. The seismograph has gain characteristics in the velocity mode of 20, the acceleration mode of 12, and the displacement mode of 200. A VS-1100D amplifier with a gain characteristic of 100 was coupled to the seismograph in all recordings. The resultant maximum gain level for velocity is 2000; acceleration, 1200; and displacement, 20,000. The three components of ground motion measured were radial, vertical, and transverse. The predominant frequencies of the site in the radial, vertical, and transverse directions are 4.5 to 75 and 25 to 37.5 Hertz.

Ambient Station 3 appears to be the most quiet location, while Ambient Station 2 appears to be the least quiet location. Results of the ambient ground motion measurements are presented on Table 2.5-16.

2.5.4.4.6 Geophysical Borehole Logging

Gamma-ray, electrical-resistance, and temperature logs were made by Dames & Moore in Borings G-2, G-6, G-8, G-12, G-14, G-15 G-16, G-18, G-19, G-21, and G-22 utilizing a Widco Porta-logger. The Widco Porta-logger is a borehole geophysical device which utilizes a variety of wireline probes in conjunction with an electronic module and recording unit. Depending on the nature of the probe used, a graphical printout of geophysical properties versus hole depth is produced.

The gamma-ray log provides a measurement of the natural radioactivity of the formation. Gamma rays are a burst of high energy electromagnetic waves which are emitted spontaneously by some radioactive elements. Nearly all of the gamma radiation encountered in the earth is emitted by Potassium 40 and the radioactive elements of the uranium and thorium series. In sedimentary formations, the gamma ray log normally reflects the shale content of the formations. Clean formations usually have a very low level of radioactivity unless radioactive contaminants such as volcanic ash or granite wash are present, or when the formation waters contain dissolved potassium salts.

The electrical-resistance of a rock depends primarily on the amount of fluid contained within the rock and its electrical resistance. The amount of fluid is a function of the porosity, therefore, the porosity of a rock is related to its resistance.

The temperature log provides a profile of the borehole temperature and can be used to locate points of fluid entry.

Results of the geophysical logging with the Widco Porta-logger are shown on the boring logs (Figures 2.5-107 through 2.5-137).

After the Widco logs were analyzed, it was decided that more comprehensive logging was necessary. Birdwell Division of Seismograph Service Corporation was contracted to run selected logs in some of the boreholes. A suit of logs consisting of gamma ray, temperature, neutron, density, caliper, and three dimensional velocity logs were run in Borings P-2, P-3, P-7, P-8, P-9, P-10, P-11, and G-21.

In addition, gamma ray and caliper logs were run in Boring G-12, G-19, and G-2 and gamma ray, temperature and caliper logs were run in Boring G-6.

The neutron log bombards the formation with high energy neutrons and measures the energy loss or slowing down of the neutrons. The greatest energy loss occurs when a neutron strikes a particle of practically equal mass, i.e., the hydrogen in the formation.

The density log bombards the formation with gamma rays, and measures the number of gamma rays returned. The number of gamma rays to reach the detector is proportional to the number of electrons in the formation and the number of electrons encountered is proportional to the bulk density of the formation.

The caliper log measures borehole diameter through the use of three independent operation measuring arms which ride the wall of the borehole and can detect variations as small as 1/4 inch in diameter. Because of the independent action of each arm, the diameter recorded is that of the circle that is described by the tips of the three arms, regardless of their radial position with respect to the center of the hole.

With the three dimensional velocity log, an omnidirectional high frequency sound pulse is generated and transmitted by refraction down the borehole sidewall for a predetermined distance where it is detected. When a full complement of elastic waves (i.e., the refracted compressional wave, composite shear wave, direct fluid wave, and boundary wave) is recorded, their velocities and the density of the borehole fluid provide the information necessary to compute the elastic moduli and densities of the rock surrounding the borehole.

The results of the Birdwell logging are shown in Figures 2.5-228 through 2.5-245 and discussed in Subsection 2.5.1.2.6.

2.5.4.5 Excavation and Backfill

Soil and rock excavations were required to reach bearing elevations of Seismic Category I structures at the site. Excavations for Seismic Category I structures at the plant site and portions of the essential service water makeup pipeline were carried into bedrock. Excavation for the river screen house was in soil.

2.5.4.5.1 Excavation and Backfill - Plant Structures

The extent of excavations and backfills for the plant structures are presented on Figures 2.5-57 and 2.5-58. A geologic section showing the plant foundation elevations is presented on Figure 2.5-59.

2.5.4.5.1.1 Soil Excavation

Soil was excavated in the plant area to expose bedrock within and a minimum of 10 feet beyond the perimeter of all rock excavations. Construction soil slopes were excavated at 1:1 to 2:1 (horizontal to vertical).

2.5.4.5.1.2 Rock Excavation

In the plant area, rock was excavated for the power block foundation, the essential service water piping, and for the essential service water cooling tower foundation. Rock was excavated by a carefully planned and monitored blasting program. This program is presented in FSAR Attachment 2.5E.

The state of stress in the in situ rock mass did not indicate evidence of the residual stresses that could cause heave. Further, the elastic rebound of the rock mass due to excavation was estimated to be less than 1/12 inch using the modulus of elasticity presented in Table 2.5-17. Therefore, there was no need for instrumentation in the main plant area to monitor heave or rebound. The excavation mapping report providing detailed information on the rock exposed in the excavated areas is presented in FSAR Attachment 2.5D.

2.5.4.5.1.3 Groundwater Control

A grout curtain was used to cut off seepage into the excavation. The grout curtain was a part of the foundation grouting program discussed in FSAR Attachment 2.5A. Minor seepage water and rainwater entering the excavation was removed using sumps. This did not have an adverse effect on the foundation material.

2.5.4.5.1.4 Backfill

Seismic Category I structures in the plant area are founded on grouted bedrock. Lean concrete was used to obtain foundation elevations in over excavated areas within the Seismic Category I structures. Within the limits of the rock excavations walls were poured directly against rock. The soil excavations were brought up to grade elevation using compacted crushed rock backfill. In areas where structures were located near the walls, the excavation was backfilled with controlled compacted crushed rock. In remaining areas, the excavation was brought up to grade using regular compacted crushed rock. The regular compacted crushed rock consisted of crushed rock similar to CA-6 material compacted to a minimum of 90% of the maximum dry density as determined by the Modified Proctor Test (ASTM 1557-70).

The controlled compacted crushed rock consists of CA-6 material compacted to a minimum of 95% of the maximum dry density as determined by the Modified Proctor Test (ASTM 1557-70). The gradation of CA-6 material is specified in the State of Illinois Standard Specifications for Road and Bridge Construction and given in Table 2.5-18. CA-6 material was obtained from rock crushing operations at the Butler Quarry located approximately 1.5 miles west of the plant site as shown on Figure 2.5-31.

Approximately 23,000 yd^3 of controlled compacted crushed rock was placed for the support of non-Seismic Category I structures in the locations shown in Figure 2.5-58. Eighty-nine in-place density tests were performed in these areas which exceeded the required 95% compaction criteria. Those areas of fill, which did not meet the compaction criteria, were recompacted or replaced. The specifications required that one in-place density test be performed for every 500 yd^3 of material placed in confined areas. The frequency of tests performed in the field was approximately one in-place density test for each 260 yd^3 of material. Representative moisture density test and particle size gradation curves for CA-6 material used as backfill are presented on Figures 2.5-77 and 2.5-78.

2.5.4.5.2 Excavation and Backfill - River Screen House

Excavating for the river screen house began in July 1977. The area was dewatered to elevation 655 by eight deep wells located around the excavation. The upper 2 to 5 feet of the excavation wall was cohesive material containing some organics. The remainder of the excavation was in a sandy gravel with pockets of fine sand. The base of the excavation was compacted with a SP-54 vibratory roller. After compaction, a BASH mudmat was poured over the entire subgrade.

Peabody Testing performed 5 in-place density tests (ASTM D-1556) on the river screen house subgrade. These tests were performed on the sandy gravel and fine sand pockets both before and after compaction. The results of these tests are listed in Table 2.5-28. The location of fine sand lenses and in-place density tests performed on the building subgrade are presented on Figure 2.5-246. Grain-size analyses (ASTM 422-63) were also performed on the sandy gravel and fine sand subgrade, and the results are listed in Table 2.5-29.

Backfilling around the river screen house began in October 1977. The excavated sandy gravel with some clay intermixed due to rehandling and CA-6 were used as backfill material.

Backfill was compacted to 95% Modified Proctor (ASTM D-1557-70). The western 50 feet of the north wall was temporarily backfilled with cohesive material in order to remove the sheet piling. This area was re-excavated in 1978, and base course and riprap were provided for slope protection.

Pittsburgh Testing performed 14 in-place density tests on the backfill around the river screen house. The results of these tests are listed on Table 2.5-30. Tests locations are shown on Figure 2.5-246.

2.5.4.5.3 Excavation and Backfill - Essential Service Water Pipeline

A construction surveillance report for the pipeline corridor is presented in FSAR Attachment 2.5G.

2.5.4.6 Groundwater Conditions

Groundwater conditions and dewatering systems during construction are presented in Subsection 2.5.4.5. Field water pressure tests and pumping tests are discussed in Subsections 2.5.4.3, 2.4.13.1.3, and 2.4.13.2.3 and are presented in Figures 2.5-104 through 2.5-140. A piezometric surface map, from which the direction of groundwater flow and the hydraulic gradient may be identified, is presented in Figure 2.4-25. A map of the piezometric surface of the Galena-Platteville aquifer is shown in Figure 2.4-24. This map shows the groundwater surface ranging from about 840 feet at the plant site to less than 740 feet, about 1 mile to the northeast. The Byron Station lies on a potentiometric high, with groundwater movement radially outward. The piezometric surface generally reflects the ground surface as expected in a water table aquifer.

Fluctuation of the water level in the Galena-Platteville Group varies depending on the time of the year the level is measured. Fluctuations of the water level in the Galena-Platteville are reported in Tables 2.4-25 and 2.4-28. Table 2.4-25 has been updated to include additional data.

Flow in the Galena-Platteville aquifer occurs along joints and bedding planes in the dolomite. Solutioning along these pathways continues at an imperceptible rate due to the low solubility of the dolomite, the hardness of the groundwater (near-saturation), and the relatively low hydraulic gradient within the aquifer. Results of chemical analyses indicate no changes in the geochemistry that may be attributed to an increase in the rate of solutioning.

Subsection 2.4.13.2.3 discusses the monitoring of groundwater levels, the past and projected fluctuations in piezometric levels, and the site vicinity groundwater flow systems.

For the design of safety-related plant structures, the groundwater level was assumed to be at plant grade, elevation 869 feet MSL, as discussed in Subsection 2.4.13.5.

2.5.4.7 Response of Soil and Rock to Dynamic Loading

2.5.4.7.1 General

A general evaluation of the dynamic properties in the various soil and rock strata encountered at the site is presented in this section. These values are based upon: (1) a review of all available field and laboratory tests performed; (2) a review of the geophysical surveys performed during these investigations; (3) a review of the latest available literature; and (4) a review of similar studies made recently for nuclear generating plants and other locations.

Dynamic tests on soil and rock samples include dynamic triaxial compression tests, shockscope tests, and resonant column tests.

Three basic parameters were developed in the dynamic studies of soil and rock. They are: (1) Young's Modulus of Elasticity (E); (2) Modulus of Rigidity (G), E and G are related by $E = 2G (1 + \mu)$ where μ is the Poisson's Ratio of the material; and (3) a damping factor.

A generalized summary of both static and dynamic moduli and damping ratios for the subsurface materials at the plant site is presented on Table 2.5-17. The dynamic moduli of elasticity and rigidity were evaluated from the results of dynamic laboratory tests and geophysical measurements. The static moduli of elasticity and rigidity were evaluated from the results of laboratory tests, a review of the latest available literature, and a comparison of the values for similar materials at other sites.

2.5.4.7.2 Laboratory Tests

2.5.4.7.2.1 Sample Preparation

To prepare intact samples for testing, the samples were carefully extruded from the sample rings and inspected for disturbance due to sampling and handling. Samples showing any visible signs of disturbance were eliminated from the testing program. A majority of the samples having appreciable gravel content or cobble fragments proved to be unsuitable for testing. Samples in which particles larger than approximately 1/2 inch were visible after being extruded, were not tested. For reconstituted samples, a preweighed amount of dry sand was vibrated into a membrane-lined

2.401-inch diameter mold in five equal layers to a height required to achieve the desired relative density.

The cap was placed on the sample and affixed to the membrane with O-rings and vacuum was applied to keep the sample from deforming while the mold (reconstituted samples) was removed and while micrometer measurements of the sample diameter and height were obtained. The triaxial cell was assembled around this sample, a small confining pressure was applied, and the vacuum was released allowing water to saturate the sample by capillary action. The required value of confining pressure was then applied with back pressure to ensure saturation. Saturation of the samples was checked by measuring Skempton's "B coefficient" (Reference 69). In all cases, the "B coefficient" was close to unity.

Cohesive soil samples, obtained from the Dames & Moore sampler were obtained to evaluate the dynamic characteristics of the cohesive soils. The samples were prepared for dynamic material property tests by first extruding the soil from the brass rings and then placing the sample in a mitre box where the ends were trimmed square. The average diameter, initial height, and weight of the sample were recorded and the sample density was calculated. The triaxial cell was assembled around the sample and the confining pressure was applied.

2.5.4.7.2.2 Dynamic Triaxial Tests

Material property tests were performed under controlled strain conditions. To begin the test a very small amplitude, 0.5 Hz sine wave signal was programmed into the loading frame. The piston was connected to the load cell, the recording equipment was zeroed, and the sample was cycled at the lowest possible strain amplitude. At the tenth load cycle, the load-deformation hysteresis loop was recorded for modulus and damping calculation The tenth load cycle was chosen for modulus and damping determination as representative of the duration of strong motion for the safe shutdown earthquake postulated for the site. At the end of cycle 25, the test was stopped. The drainage valve was opened, and excess pore water pressure was allowed to dissipate. The drainage valve was again closed, a new slightly higher strain amplitude was programmed, and another test was performed. This procedure was repeated 6 or 7 times for each sample giving a record of dynamic sample response covering the range of approximately 0.01 to 1.0% single amplitude axial strain.

Values of dynamic Young's modulus were determined by measuring the slope of the line connecting the extreme points of the hysteresis loops obtained at the tenth load cycle. The same loop was used to calculate the hysteretic damping, using the equation:

$$\lambda = \frac{1}{2\pi} \; \frac{\Delta W}{W} \qquad (2.5\text{-}1)$$

where: ΔW is the total dissipated energy per cycle as represented by the area of the hysteresis loop and W is the work capacity per cycle.

The value of Poisson's ratio required for these calculations is generally estimated because of the fact that accurate measurements of Poisson's ratio are difficult to determine experimentally.

For cohesionless soils, the modulus has been found to be related to the confining pressure by the following equation:

$$G = 1000\ K_2\ (\bar{\sigma}_m)^{1/2} \tag{2.5-2}$$

where: K_2 is a soil parameter and $\bar{\sigma}_m$ is the mean effective stress which for triaxial tests under isotropic conditions is equal to the effective confining pressure. Thus, the influence of strain amplitude on the modulus can be expressed through its influence on the parameter K_2.

For cohesive soils the wide variation in dynamic soil properties are often taken into account by normalizing the shear modulus (G) with respect to the undrained shear strength (Sμ) and expressing the relationship G/Sμ as a function of shear strain.

The test results are presented in Table 2.5-19 for the plant site soils with the dynamic shear modulus versus shear strain shown on Figure 2.5-83 and the damping curve is shown on Figure 2.5-84. The corresponding data for the river screen house are shown on Figure 2.5-249 which indicates the results of the cyclic triaxial tests on both intact and recompacted samples plotted in a normalized form. The foundation for the river screen house was placed on natural soils, and the river screen house, therefore, is not resting on recompacted material. The points plotted on this figure were established by anchoring the shear modulus (G) at the lowest strain level obtained during the triaxial tests on the Seed and Idriss (Reference 97) normalized shear modulus (G/G_{max}) strain degradation curve. By obtaining the normalized shear modulus value at this strain level, a G_{max} value for the tests was obtained based on the mean Seed and Idriss (Reference 97) strain degradation curve for strains smaller than the minimum shear strain for the tests. The other data points were then normalized based on the obtained G_{max} value.

The results of normalizing the test data show that the undisturbed samples exhibit strain degradation characteristics within the range postulated by Seed and Idriss (Reference 97), however, the recompacted soil samples follow strain degradation curves unreasonably steep compared to the normalized curve. These characteristics are the result of the test procedures rather than actual soil properties. The tests on the recompacted samples, completed approximately 1 year prior to the tests on the intact samples, were performed on a machine that had been calibrated according to standard procedures, but not corrected for piston friction. A correction for piston friction is usually not necessary for property tests at high strain values where high loads are required to obtain the desired deformation. At small strains, where smaller loads are required, the piston friction represents a significant portion of the recorded load. It should be noted that data from triaxial tests were at the time of the tests considered reliable only for shear strains greater than 0.01%. Regulatory Guide 1.138 shows that the strain range for the cyclic triaxial test is limited to shear strains greater than 0.01%. The calculated moduli at lower strains are, therefore, larger than the true values, which results in an apparent very steep strain degradation relationship from these incorrectly high shear moduli for low strains. The curves for the reconstituted samples are in error at low strains and are not representative of the granular material underlying the river screen house. This conclusion is supported further by more recent test results (Reference 98) which show that the strain degradation curves for granular material may be flatter than those suggested by Seed and Idriss.

The test data on undisturbed samples are presented in the form of normalized shear modulus factor K_2 versus strain on Figure 2.5-250. The resonant column data were recorded at a shear strain on the order of 10^{-4} percent and, thus, represent anchor points. The data indicate normalized shear modulus factors in the range of 40 to 85.

The shear modulus factors for the deposits underlying the river screen house were also evaluated using the empirical expression given by Hardin and Drnevich (Reference 94). The average K_2 obtained using this procedure was 68 (Figure 2.5-251). However, Hardin (Reference 95) has proposed that the shear modulus for granular material is also a function of grain size, in particular the particle size at which 5 percent of the sample is finer (D_5). Using the procedure proposed by Hardin and a D_5 of 0.2 $\frac{5}{mm}$ (see Figure 2.5-49), a shear modulus factor of 75 was obtained. Thus, the K_{2max} values obtained using the empirical relationships proposed in References 94 and 95 (68 to 75) fall within the range (65 to 90) given in Figure 2.5-89.

In addition to using the empirical relationship proposed by Hardin and Drnevich, the shear modulus factor for the deposits underlying the river screen house were evaluated using the procedures proposed by Ohsaki and Iwasaki (Reference 96). This procedure is based on an empirical relationship between dynamic shear modulus as determined from field measurements and standard split spoon resistance (SPT). This method has been used for sandy and gravelly soils successfully, and therefore is applicable to sandy and gravelly soils at the river screen house. Thus, using the SPT values at the river screen house, shear modulus factors in the range of 57 to 122 were obtained, with a mean shear modulus factor of 85 (Figure 2.2-251). It should be noted, however, that evaluation of field data (Reference 99) indicates that the procedure proposed by Ohsaki and Iwasaki generally overestimates the field shear modulus by approximately 25 percent.

The shear modulus factors obtained for the materials underlying the river screen house were also compared with those calculated from field data obtained at other sites (Reference 93). The results, shown in Table 2.5-35 and Figure 2.5-252, show shear modulus factors in the range of approximately 40 to 100 for sites with penetration resistance similar to those encountered under the river screen house.

The shear wave velocities presented at other sites (Reference 93) were also plotted versus mean standard penetration resistance. The results are shown on Figure 2.5-253 together with the shear wave velocity calculated using the shear moduli obtained from the procedures of Ohsaki and Iwasaki. The data plotted in Figure 2.5-253 show that the procedures proposed by Ohsaki and Iwasaki are in good agreement, although close to an upper bound, with field data presented by Shannon and Wilson (Reference 93).

Figure 2.5-254 shows calculated shear wave velocities versus depth based on a wide range of normalized shear moduli. Also shown are the shear wave velocities corresponding to the shear moduli obtained using the empirical relationships proposed by Hardin and Drnevich (Reference 94), Hardin (Reference 95), and Ohsaki and Iwasaki (Reference 96). Based on the empirical relationships, the shear wave velocity at the site of the river screen house varies between approximately 750 and 1600 fps. These velocities are in good agreement with the field data presented by Shannon and Wilson (Reference 93) and Figure 2.5-253 which show shear wave velocities in the same range for sites with similar standard split spoon penetration resistances to those encountered under the river screen house at Byron.

A seismic crosshole survey was conducted adjacent to the location of the river screen house to obtain seismic shear and compressional wave velocities of soils representative of those underlying the screen house. A report presenting procedures and results of the survey is given in Byron FSAR Attachment 2.5J. Figure 2.5-255 shows measured shear wave velocities versus depth. These velocities range between 750 and 1600 fps as predicted. Figure 2.5-256 shows the shear modulus factor K_{2max} versus depth based on the measured shear wave velocity. The data presented show a mean value of K_{2max} of 79. This value is within the range (K_{2max} between 65 and 90) shown in Figure 2.5-89.

Results of K_{2max} obtained from calculations based on empirical relationships (References 94, 95 and 96) and geophysical measurements at Byron river screen house and at other sites indicate that the values presented in Figure 2.5-89 are reasonable and correct for the materials underlying the river screen house.

The variation of shear modulus (G_{max}) at low strain levels (10^{-6} inch/inch) calculated from the measured shear wave velocities given in Figure 2.5-255 is presented in Figure 2.5-257. The cumulative average with depth of G_{max} indicates that G_{max} varies from a low of 2794 ksf at a 25-foot depth to a high of 6104 ksf at a 110-foot depth.

Figure 2.5-258 presents a shear modulus versus shear strain variation determined by anchoring the field mean shear modulus value (G_{max}) at 1×10^{-4} percent strain and matching the mean Seed and Idriss strain degradation curve to obtain a shear modulus variation at higher strains. Plotted on this curve is the shear modulus variation used in the analysis. The shear strain range corresponding to these values is approximately 1×10^{-1} percent to 4×10^{-1} percent. This range generally falls within the range expected for strong motion earthquakes (Regulatory Guide 1.138).

2.5.4.7.2.3 Shockscope Tests

Compressional wave velocity (shockscope) tests were performed on representative rock samples by Professor M. L. Silver in the Soil Mechanics Laboratory at the University of Illinois, Chicago Circle Campus. The velocity observed in the laboratory was used to correlate field velocity measurements obtained during the geophysical survey.

In the test, samples are subjected to a physical shock, and the time required for the shock wave to travel the length of the sample is measured. The velocity of compressional wave propagation is then computed. The samples were tested in an unconfined state. The test results are presented in Table 2.5-21.

2.5.4.7.2.4 Resonant Column Tests

Dynamic torsional shear (resonant column) tests were performed on representative soil and rock samples to evaluate the modulus of rigidity of these materials. The method of performing resonant column tests is described on Figure 2.5-85. The tests were conducted on solid cylindrical samples at natural moisture content and density over a range of confining pressures. The results of the resonant column tests are presented in Table 2.5-22.

2.5.4.7.3 Soil-Structure Interaction

Soil-structure interaction analyses are presented in Subsection 3.7.2.4.

2.5.4.7.4 Buried Pipelines

Seismic analysis of buried pipelines is provided in Subsection 3.7.3.4.

2.5.4.8 Liquefaction Potential

2.5.4.8.1 General

The river screen house and portions of the essential service cooling water makeup line adjacent to the river screen house are founded on granular soils. Other Seismic Category I structures are founded on bedrock. Liquefaction analysis for the river screen house and portions of the makeup pipeline is presented in the following subsections. This liquefaction evaluation was initially performed based on an artificial base rock motion selected to produce the standard NRC broad band spectra scaled to maximum horizontal ground acceleration of 0.12g at the bedrock level. For the purpose of licensing, the liquefaction analyses was reperformed based on two real earthquake time-histories scaled to 0.20g and applied at the bedrock surface.

2.5.4.8.2 Simplified Analysis

Initially the liquefaction potential of the granular soil deposits in the vicinity of the river screen house was evaluated on the basis of the simplified procedure described by Seed and Idriss (Reference 70). This procedure is based on both theoretical considerations and descriptions of site conditions where liquefaction is known to have occurred or not to have occurred under earthquakes of known or estimated magnitudes.

The liquefaction potential evaluation chart, as proposed by Seed and Idriss, is shown on Figure 2.5-86. A maximum horizontal ground acceleration of 0.20g was used for the simplified analysis.

The standard penetration resistance values obtained from the borings drilled at the river screen house are shown superimposed on the liquefaction potential evaluation chart on Figure 2.5-86. The relationship between relative density and blow count was evaluated on the basis of Gibbs and Holtz, (Reference 71). The majority of the measured standard penetration resistance values indicate that liquefaction under the postulated safe shutdown earthquake is unlikely at all depths throughout the alluvial deposit. However, the margin of safety against liquefaction was further investigated as described in Subsection 2.5.4.8.3.

2.5.4.8.3 One Dimensional Wave Propagation Analysis

The liquefaction potential of the alluvial deposit underlying the river screen house during the safe shutdown earthquake was further evaluated in accordance with the procedures described by Seed and Idriss, (Reference 72), and Seed and Peacock, Reference 74.

A series of laboratory liquefaction tests was performed on representative samples of the alluvial soils to determine the number of cycles of load application required to produce liquefaction at various levels of dynamic shear stress. The computer program SHAKE was used for the one dimensional wave propagation analysis. The soil layers are shown together with soil unit weights and the effective stresses for each layer in Table 2.5-33. The soil parameters were based on the effective stresses in Table 2.5-33 and the upper and lower bounds of properties presented in Figure 2.5-89.

The failure criterion used for the liquefaction tests was adapted based on the 1971 San Fernando earthquake studies which were published by H. B. Seed et al., 1975.

The failure criteria and deformation of laboratory samples are not directly related to in situ strain conditions. However, since the liquefaction analysis shows that no liquefaction will occur, the maximum seismically induced movement of the river bank was evaluated based on the procedure outlined by Newmark (1965) and Makdisi and Seed (1978). The calculations show the deformations to be 0.1 inch or less. This will have no effect on the site structures.

2.5.4.8.3.1 Dynamic Soil Parameters

The soil parameters for dynamic response analysis were selected on the basis of density determinations, and resonant column and dynamic triaxial tests.

The standard penetration resistance values, as shown on the logs of borings (Figures 2.5-138 and 2.5-186 through 2.5-189) were erratic, reflecting the heterogeneous and interbedded nature of the deposit as indicated by the large variation in the grain size distribution (especially gravel content) throughout the deposit. Because of the influence of the gravel content on the measured standard penetration resistance values, the variation in these values are not considered to be representative of variations in the relative density.

The results of the density determinations based on laboratory moisture and density tests of samples obtained from the borings also produced large variations. The results of the laboratory determinations of in-place wet density and moisture content of samples obtained by a variety of samplers are presented on Figures 2.5-230 and 2.5-231.

Samplers used in the field sampling program were 2.4-inch diameter Dames & Moore Type U samplers; 3-inch diameter piston, Denison, Pitcher and Shelby tube samplers; 4-inch diameter core samplers; and 5- and 6-inch diameter Shelby tube samplers. It was found that Shelby tube samplers produced the best results among all samplers used, insofar as field penetration and sampler recoveries were concerned. No significant difference is noted among the results obtained from samples taken by various samplers.

The in situ densities used in the analysis are given in Table 2.5-23. They are approximately the lowest values found during testing. The actual test results are found on Figures 2.5-87 and 2.5-88. The use of these values in the liquefaction analyses will yield more conservative results.

The results of laboratory resonant column and dynamic triaxial compression tests performed on intact and reconstituted samples are summarized on Tables 2.5-22 and 2.5-20, respectively and in Subsection 2.5.4.7.2. The values of the dynamic shear modulus and damping ratio, as a function of shear strain level, are shown on Figures 2.5-89 and 2.5-90, respectively. The procedures for obtaining and preparing samples and conducting the dynamic laboratory tests are described in Subsection 2.5.4.7.2.

In order to assess the effect of variations of the dynamic soil parameters on the dynamic response of the deposit, lower-bound (Case 1) and upper-bound (Case 2) values of the measured dynamic shear moduli were selected for analysis as shown on Figure 2.5-89. In the selection of the upper- and lower-bound values, the test results obtained from reconstituted samples were eliminated from consideration for the following reasons:

a. A large difference was noted between the dynamic behavior of reconstituted samples compared to intact samples at similar densities. The intact samples are considered to be more representative of the in situ structure of the granular deposit since particle segregation is difficult to prevent during reconstituting samples and reconstruction of a packing representative of the in situ condition may not be obtained. Comparing the test results of the reconstituted samples versus the intact samples also shows consistently higher shear moduli for the reconstituted samples, as shown on Figure 2.5-89. Therefore, it was concluded that reconstituting the samples altered the dynamic properties of the material.

b. Since all liquefaction tests were performed on intact samples, the dynamic response and liquefaction behavior should be based on test data obtained from intact samples prepared in the same manner.

The solid curve shown on Figure 2.5-90 was used to represent the damping characteristics for both Case 1 and Case 2. This curve was selected as a reasonable estimate of the damping characteristics as measured on the intact samples.

2.5.4.8.3.2 Dynamic Response Analysis

Initially the base rock motion used in the dynamic response analysis was that of an artificial earthquake of 10-second duration, selected to essentially envelope the standard NRC broad band spectra scaled to a maximum horizontal ground acceleration of 0.12g at the bedrock level. The horizontal acceleration time history for the artificial base rock motion is presented on Figure 2.5-91. The corresponding response spectra, compared to the NRC broad band response spectra are shown on Figures 2.5-93 through 2.5-95 for various damping ratios.

Subsequently two other base rock motions were used in the dynamic response analysis. These consist of the first 10-second duration of two real earthquakes; namely, the S80E component of the March 22, 1957 Golden Gate Park earthquake and the N-S component of the 1935 Helena, Montana earthquake. The horizontal acceleration time histories for the two earthquakes are presented on Figure 2.5-92, scaled to a maximum acceleration of 0.20g. The corresponding response spectra are shown on Figures 2.5-96 through 2.5-99 for various damping ratios.

The dynamic response of the alluvial deposit was evaluated by means of a wave propagation analysis of a one-dimensional model of the deposit. The one-dimensional model is considered appropriate for this deposit since the ground surface and bedrock surface have an average slope of less than 5% for several hundred feet in all directions. The deposit was subdivided into a series of 14 discrete horizontal layers, each characterized by appropriate values of the relevant soil parameters, including unit weight, shear modulus, and damping ratio.

The one-dimensional layered system was subjected to the artificial base rock motions shown on Figure 2.5-91 for the two sets of dynamic soil parameters described previously. The numerical analysis was performed by a digital computer using the wave propagation technique. Several iterations were performed to achieve compatibility between the shear strains in each individual sublayer and the corresponding dynamic shear moduli and damping ratios. The shear stress time histories for the middle of the individual sublayers were computed from the strain compatible solutions.

Since the analyses utilizing the upper-bound dynamic soil moduli produced higher shear stresses, only the upper-bound value of the dynamic soil properties (Case II) were utilized in the one-dimensional wave propagation analysis for the two base rock motions of the two real earthquakes (Figure 2.5-92).

2.5.4.8.3.3 Laboratory Liquefaction Tests

A series of laboratory dynamic triaxial tests were performed on intact samples obtained from the borings using a Dames & Moore Type U Sampler. As discussed previously in Subsection 2.5.4.8.3.1, the testing of intact samples was considered to be the most representative and most consistent method of determining the dynamic behavior of in situ soils.

Samples were prepared in accordance with Subsection 2.5.4.2.1. Samples prepared in this manner inevitably involve some degree of disturbance due to sampling and handling. However, since the Standard Penetration Test results indicate that the relative density is generally in excess is generally in excess of 60%, it is believed that any disturbance of this material will cause a reduction in density, resulting in conservative test results.

Liquefaction tests were performed under controlled stress conditions. A sine wave, 1 Hz load signal with an amplitude equal to the specified cyclic vertical stress was programmed into the load frame. The load was cycled until the cyclic single amplitude vertical strains reached 10%.

A total of 11 cyclic triaxial liquefaction tests were conducted at several different values of confining pressure and cyclic vertical stress. A summary of the liquefaction tests is presented in Table 2.5-24. The criterion for liquefaction of the laboratory samples was defined as the number of cycles required to produce single amplitude shear strain equal to 5%.

The results of the liquefaction tests are shown on Figure 2.5-100 in the form of stress ratio (ratio of one-half of the cyclic vertical stress to confining pressure) versus the number of stress cycles required to produce liquefaction, plotted on a semilogarithmic scale. Figure 2.5-100 shows that the liquefaction potential varied considerably depending upon the density and grain size characteristics of the individual samples. The solid curve on Figure 2.5-100 represents the mean value of the test results determined by the method of least-squares.

2.5.4.8.3.4 Evaluation of Liquefaction Potential

The potential for liquefaction of the granular soils at the site of the river screen house was evaluated by comparing the shear stress time-history computed at the middle of each of the individual sublayers with the shear stress required to produce liquefaction under field conditions. It is assumed that the groundwater is at the surface.

Because of the difference between field and laboratory stress conditions and the limitations in testing equipment and procedures, the cyclic shear stress required to produce liquefaction in the field is considered to be 65% of the laboratory values (Seed and Peacock, Reference 73). On this basis, the stress ratio required to produce liquefaction under field conditions is represented by the dashed curve shown on Figure 2.5-100.

In these studies, the liquefaction potential of the alluvial deposit was evaluated using the "cumulative average" procedures.

In the cumulative average procedure, the peak of the time histories of shear stresses, for each of the individual sublayers were sorted and evaluated in the following manner:

a. Sorting of the peak shear stresses for each time-history, saving only the largest stress value between each zero crossing (both positive and negative peak values are saved).

b. Rearranging of the peak shear stresses in descending order so that the largest peak is first and the smallest is last. It is assumed that, for practical purposes, the sequence in which the shear stresses occur does not affect the final answer.

c. Averaging of the absolute values of the positive and negative stress peaks for each cycle.

d. Computing the cumulative average shear stress for increasing numbers of cycles. The average value corresponding to any N number of cycles is compared to the laboratory test data to evaluate the significant number of cycles and the factor of safety against liquefaction at each depth.

A summary of the results obtained using this procedure is presented on Table 2.5-25 for the artificial base rock motion. This tabulation indicates that liquefaction will not occur at any depth within the deposit. The minimum factor of safety below the base of the foundation is equal to 1.54.

For the base rock motions using the actual Golden Gate Park and the Helena, Montana earthquake time histories, the results of liquefaction analyses using the cumulative average procedure are summarized on Table 2.5-26. Again, the results indicate that liquefaction will not occur at any depth within the deposit and the minimum factor of safety below the base of the foundation is approximately equal to 1.7.

The liquefaction potential at the river screen house was reevaluated in the depth interval between 50 and 65 feet using the procedures proposed by Seed (1976). A surface wave magnitude earthquake of 5.7 with a maximum ground surface acceleration of 0.2g was used for the liquefaction analysis. The SPT penetration resistance was normalized based on confining pressure, and the mean blow count less one standard deviation was used for the evaluation of the liquefaction potential.

The results of the analyses give a factor of safety against liquefaction of 1.75 in the depth interval 50 to 58 feet and 1.56 between 58 and 65 feet.

The potential for liquefaction of the granular material at portions of the essential service water makeup pipeline was evaluated by similar procedures used for the river screen house.

The shear modulus of the soils underlying the pipeline were estimated utilizing the data in Figures 2.5-83 and 2.5-89 and mean effective over-burden soil pressures (Table 2.5-34).

In design of the buried piping, the variability of the supporting soil strata has been accounted for by conservatively choosing the design particle velocity and the apparent shear wave velocity.

Recent studies (Chaney 1978, and Martin et al., 1978) show that the resistance to liquefaction increase substantially following reduction of the degree of saturation to levels below 99%. Chaney states that a degree of saturation in excess of 99% must be achieved before liquefaction occurs in less than 1000 cycles. Martin et al., shows that the stress ratio required to cause liquefaction in 10 cycles for loose sand (relative density 45%) increases by approximately 100% to 200% when the degree of saturation decreases from 100% to 99% and 98%, respectively.

The groundwater table was not encountered within the soils which underlie the pipeline in Areas of Concern Nos. 11 and 12. The moisture content determined by testing samples obtained during the investigation along the pipeline range from 2.5% to 17.4% with a mean value of 10.6%. Assuming a minimum void ratio of 0.60 for the loose sands, this moisture content corresponds to a mean degree of saturation of 47%. Air filled pore space, therefore, makes up approximately 20% of the soil matrix, i.e., for a 10-foot thick deposit to become saturated, a water column of 24 inches must infiltrate and remain in the soil.

Since liquefaction will not occur, the settlement caused by seismically induced loads was calculated based on Silver and Seed, 1971, and Pyke et al., 1975. The estimated maximum settlements, according to these procedures, in Areas of Concern Nos. 11 and 12 are 1.5 and 0.5 inches, respectively.

Since the soil profiles in the sections in question appear to be relatively homogeneous with respect to permeability characteristics, i.e., obvious impervious layers were rarely encountered in the borings, and since the bedrock contains numerous joints and fractures, most of the water that infiltrates the soils along the pipeline route should be quickly lost by percolation through the near surface soils and joints and fractures in the bedrock. During summer months, transpiration and evaporation will account for additional loss of soil moisture. Therefore, it appears that the geohydrological conditions in the area are not conducive to the development of perched water conditions or saturation of the subsurface soils. The soils underlying the pipeline, therefore, are not susceptible to liquefaction.

2.5.4.8.4 Summary of Liquefaction Analyses

The results of the liquefaction studies indicate that the granular deposit in the vicinity of the river screen house will not liquefy when subjected to the safe shutdown earthquake. The laboratory liquefaction tests were conducted under undrained conditions so that excess pore pressures could not dissipate during the tests. Since the deposit is relatively coarse grained, it is anticipated that the coefficient of permeability in situ would be large enough to allow considerable pore pressure dissipation during earthquake loading under field conditions. Permeability coefficients estimated from the effective grain sizes indicate that approximately 90% of the sand samples show permeability coefficients on the order of 0.1 centimeters per second, while no samples having permeabilities of less than 5 x 10^{-3} centimeters per second are present. A survey of the available literature also indicates that there are no known cases of liquefaction of similar coarse granular materials for the earthquake postulated for this site.

2.5.4.9 Earthquake Design Basis

2.5.4.9.1 General

This subsection provides a summary of the derivation of the OBE and SSE, and a summary of the earthquake selection for liquefaction.

2.5.4.9.2 Safe Shutdown Earthquake (SSE)

A detailed discussion of the SSE can be found in Subsection 2.5.2.6.

The recommended safe shutdown earthquake was defined as the occurrence of an Intensity VII event near the site. This near field event would produce maximum horizontal ground accelerations of 0.12g (Reference 74 and Figure 2.5-39).

For the purpose of licensing, a horizontal ground acceleration of 0.2g was selected for the SSE. This would be equivalent to the occurrence of a Modified Mercalli Intensity VII-VIII (Reference 74). As an additional means of conservatism, this value had been applied at foundation level. Utilizing the subsurface properties presented in Subsection 2.5.4.7, the corresponding ground surface acceleration was found to be 0.21g.

2.5.4.9.3 Operating Basis Earthquake (OBE)

A detailed discussion of the deviation of the OBE is given in Subsection 2.5.2.7.

The operating basis earthquake is intended to indicate those levels of ground motion which could reasonably be expected to occur at the site.

On the basis of the seismic history of the area, it appears unlikely that the site will be subjected to any ground motion of significant levels during the life of the nuclear power station It is probable that the maximum level of ground motion experienced at the site during historic time was Intensity VI and was due to the 1909 Intensity VII, Beloit earthquake. For this condition, the maximum horizontal ground acceleration on rock at the site was probably on the order of 0.06g.

A probability analysis (Reference 67) of the occurrence of earthquakes at the Braidwood Station was performed using the data on past earthquakes in the area and the available information on the attenuation of intensity over the distance between the earthquake location of the site. The results of this probability analysis are also applicable to the Byron Station and show that a site Intensity of MMI VI on the Modified Mercalli scale has an average return period of 2150 years. Because of this long return period, the site Intensity of VI was selected conservatively as the operating basis earthquake. However, for the purposes of licensing of the plant, the acceleration level for the OBE was selected as 0.09g.

2.5.4.9.4 Earthquake Selection for Liquefaction and Seismic Response Analysis of Earthworks

Liquefaction analysis was performed only for the river screen house foundation soils and portions of the essential service water makeup pipeline since the main plant is founded on rock. The justification of the earthquake selections for the analysis can be found in Subsection 2.5.4.8.

The results show that the granular deposits in the vicinity of the river screen house will remain stable under the postulated SSE.

2.5.4.10 Static and Dynamic Stability

2.5.4.10.1 General

The stability of all safety-related structures has been analyzed for static and dynamic loading conditions using conservative analytical procedures and subgrade material properties. Foundation data for major structures is presented in Table 2.5-27. A discussion of the analyses is presented in the following sections.

2.5.4.10.2 Plant Structures

2.5.4.10.2.1 General

All Seismic Category I plant structures are supported on mat foundations on bedrock. Other major plant structures are supported on mat foundations established on bedrock and/or controlled compacted crushed rock with gradation as shown on Table 2.5-18. The foundation bedrock has been grouted to top of the Harmony Hill Shale Member of the Glenwood Formation approximately 200 feet below the natural bedrock surface as shown on Figure 2.5-170. The major grout communication pattern followed the major northwest-southwest joint pattern in the bedrock which is documented by field investigations, excavation mapping and aerial photograph interpretation (FSAR Attachments 2.5A and 2.5D and Figure 2.5-101).

As shown in Figures 2.5A-5 through 2.5A-8, the quantity of grout pumped into the Dunleith was about 1% of the total grout pumped during the foundation grouting program. The formations of the underlying Platteville Group consumed over 95% of the grout which was injected at depths of over 100 feet below the bedrock surface. Joint sets are common to the Galena and Platteville groups, however, the results of the grouting program indicate that the wide vertical separation between the zone of the high grout take and the sinkhole in the upper bedrock do not support a significant correlation. Detailed inspection of the pipeline corridor was made during its construction and no conditions were found or could be reasonably hypothesized by extension along the joint trends to indicate significant design impacts.

The subsurface investigations (examination of rock cores, mapping of outcrops, and performance of borehole geophysics and caliper logs) performed at the site indicated that solution activity had occurred and that the lower limit of solution activity is stratigraphically controlled by the Harmony Hill Shale Member of the Glenwood Formation. Therefore, it was concluded that the foundation bedrock should be grouted to the top of the Harmony Hill to assure that no significant voids are present below the foundations of Seismic Category I structures. The purpose of the grouting program (FSAR Attachments 2.5A and 2.5B) was to fill voids, seal off solution channels, and solidify the mass below these foundations.

The investigations performed to determine if any significant cavities exist after grouting were: 1) a detailed evaluation of the grouting program which included primary grout holes spaced at a 20-foot grid pattern around and across the foundation; and 2) verification borings. The results of these investigations are presented in FSAR Attachments 2.5A and 2.5B.

These investigations and evaluations provided the basis that the grouting program was successful and that all significant solution features below the foundations were grouted.

2.5.4.10.2.2 Bearing Capacity

The ultimate bearing capacity of the foundation bedrock was evaluated on a conservative basis, in accordance with methods described in Stagg and Zienkiewicz (Reference 75). No consideration was given to the increase in bearing capacity which will result from the grouting.

The strength of the foundation rock was evaluated by means of rock compression tests prior to grouting. Considering this value to be appropriate for rock with an RQD (rock quality designation) of 100%, a reduction factor was selected based on an assessment of the measured RQD values, information on vug volume and size, fracture orientation and spacing, and presence of clay and shale seams. The test strength was modified using this reduction factor to determine the representative in situ strength of the rock mass. On this basis, the minimum ultimate bearing capacity of the rock mass used in the plant area prior to grouting was 200,000 psf. Therefore, for a maximum static loading of 10,000 psf, the factor of safety against foundation failure is 20. Under the postulated SSE event, the maximum combined static and dynamic loading is estimated to be 35,000 psf. The factor of safety for this condition is in excess of 5. The factor of safety would be considerably in excess of the calculated values if the rock strengthening due to grouting was considered.

Structures resting partially or entirely on the compacted crushed rock include the turbine buildings, radwaste building, service building, and heater bay. The ultimate bearing capacity of the compacted fill was evaluated, on a conservative basis, in accordance with methods described in Terzaghi and Peck, (Reference 76).

On this basis, the ultimate bearing capacity is in excess of 20,000 psf. For a maximum static loading of 4000 psf, the factor of safety against foundation failure will exceed 5.

2.5.4.10.2.3 Settlement

The total settlement for the structures on the foundation bedrock (reactor containment, auxiliary building, fuel handling building, essential service cooling tower, essential service water area, turbine-generator pedestal, and most of the turbine building) was calculated to be less than 1/8 inch using the method described by Coates (Reference 77) and the rock properties presented in Table 2.5-17. The total settlement for the structures founded entirely on compacted crushed rock (radwaste building and service building) is estimated to be on the order of 1/4 inch. Differential settlements on the order of 1/4 inch may occur within all structures founded partially or entirely on compacted crushed rock.

The Seismic Category I plant structures are all founded on grouted bedrock. The total and differential settlements calculated for the bedrock for static and SSE conditions were based on the elastic moduli of the dolomite. The results of the calculations showed negligible total and differential settlement. The solution channels and sinkholes will not affect the settlement because the grouting program has filled these zones with grout.

As a part of the project settlement monitoring program shrinkage cracking was observed on the two longitudinal short walls between the cooling towers and on the end transverse walls. Two types of cracks were observed on the longitudinal short walls: shrinkage cracks affecting the surface of the concrete; and cracks at the construction joints between the short walls and the end transverse walls. The shrinkage cracks are narrow, less than 0.001

inch, and random in nature. The construction joint cracks are caused by the seasonal thermal movement of the end transverse walls. The end short walls are not part of the lateral load resisting system. At the end transverse walls, vertical cracks were observed. These are shrinkage and thermal cracks. Horizontal cracks in these walls, at about 2 feet from the top surface of the mat foundation, are thermal cracks caused by the contraction of the hot water basin. Both the shrinkage and the thermal cracks are on the order of 0.001 inch wide or less on the outside surface of the end transverse walls and on the longitudinal short walls, some diagonal cracks and wedge shaped cracked concrete were observed. These are localized cracks caused by the alternating thermal movements of the end transverse walls. These localized cracks do not affect the structural resisting capability of the cooling tower. Additionally, the towers are founded on rock, and no settlement pattern of cracks can be seen.

Shrinkage cracks and horizontal thermal cracks are typical of structures the size of these cooling towers. The observed crack widths are less than the limiting crack width of 0.013 inch allowed in Section 10.6 of the ACI 318-77 "Commentary on Building Code Requirements for Reinforced Concrete." The cracks observed do not reduce the structural resistance capability of the towers. Therefore, no structural repair is needed. Spalled concrete at the construction joint will be repaired and a flexible sealer will be placed at the joint between the longitudinal short walls and the end transverse walls.

Rebound of Seismic Category I structure foundations is discussed in Subsection 2.5.4.5.1.2.

2.5.4.10.3 River Screen House

2.5.4.10.3.1 General

The river screen house is supported on mat foundations established on the natural soils. The base of the foundation mat at elevation 660 feet is approximately 12 feet below the groundwater table and approximately 22 feet below the design flood stage. A generalized subsurface profile of the river screen house is presented on Figure 2.5-60.

2.5.4.10.3.2 Bearing Capacity

The ultimate bearing capacity of the mat foundation for the river screen house was evaluated in accordance with the method described by Terzaghi and Peck (Reference 76). The strength of the foundation soil (angle of internal friction) was evaluated from density data, grain size distribution, and standard penetration resistance. This information is presented in Subsections 2.5.4.2.2.1 and 2.5.4.3.2.3. On this basis, the angle of internal friction of the granular foundation soil is considered to be at least 38°. Allowing no increase in bearing capacity for embedment of the foundation, the minimum ultimate bearing capacity of the river screen house foundation is on the order of 100,000 psf. For a maximum static loading of 2000 psf, the factor of safety against foundation failure is 50. Under the postulated SSE event, the maximum combined static and dynamic loading is 4500 psf. The factor of safety for this condition is in excess of 22. The factor of safety would be considerably in excess of the calculated value if the additional bearing capacity resulting from partial embedment of the structure was considered.

2.5.4.10.3.3 Settlement

The total static settlement of the river screen house has been estimated to be 1 inch using the Janbu method (Reference 78). This analysis was performed using a modulus number, m, of 500 and a stress exponent, a, of 0.5. The settlement of the river screen house will be substantially completed during construction because granular soils such as those present at the river screen house settle rapidly in response to load. The settlement of the river screen house has been monitored using seven settlement monuments established in August and December, 1977. The settlement records taken to date indicate that settlement of the river screen house is completed (Subsection 2.5.4.13).

Dynamic settlements from a safe shutdown earthquake were computed according to Pyke (Reference 79) and the data Pyke obtained from the Monterey No. O sand, compacted to 70% relative density. The additional settlement caused by a safe shutdown earthquake has been estimated as 1.3 inches for the 115 feet of sand at the river screen house. Differential settlement would be less or at the most equal to the total settlement of 1.3 inches.

2.5.4.10.4 Essential Service Water Makeup Pipeline

Settlement and bearing capacity is not a concern for the static stability analysis of the essential service water makeup pipeline. The net increase in soil loading due to the pipeline (weight of pipeline minus the weight of soil it displaced) is not significant from an engineering viewpoint. The makeup line is buried in the natural soils and bedrock formations at a minimum depth of 5 feet below the ground surface to prevent freezing or frost heave.

Occasional small sinkholes have developed in the dolomite formations between the bluff-line and the main plant and can be identified as being post-Pleistocene and pre-Pleistocene to Wisconsinan in age, Figure 2.5-101. The maximum diameter of the post-Pleistocene aged sinkholes is approximately 50 feet with the average being approximately 25 feet. It is highly improbable that additional sinkholes will develop along the makeup line route during the life of the plant. However, even if it is assumed that the sinkholes may develop, the area affected would be on the order of 25 feet with a possible maximum development of

50 feet. The makeup line is designed to span any sinkhole of diameter up to 50 feet.

The design-basis sinkholes was chosen according to the age of the sinkholes. Sinkholes which may form or have formed in the present (Holocene) were used for the design of the makeup line. There was only one sinkhole present in the site area with a greater diameter than 50 feet, the design-basis sinkhole. This sinkhole, which has a diameter of 150 feet, has 31 feet of Wisconsinan and residual soil deposits overlying the bedrock surface at the approximate center. The age of this sinkhole, based on the thickness of the Wisconsinan and residual soil deposits, makes it pre-Pleistocene to Wisconsinan in age. The thickness of Wisconsinan and residual soil deposits is thicker over the than the Wisconsinan and residual soil deposits sinkhole overlying the bedrock beyond the perimeter of the sinkhole, see Figure 2.5-102. The other sinkholes present in the site area, diameters up to 50 feet, are located in areas where there are little or no Wisconsinan and residual soil deposits so that it is not possible to put a relative age on them. On this basis, a 50-foot diameter sinkhole has been selected as the design sinkhole.

2.5.4.10.5 Lateral Pressures

Subsurface walls were designed to resist both the static and dynamic pressures resulting from the surrounding earth and water. Design lateral pressures for subsurface walls at Byron Station (except for the river screen house and the essential service water cooling towers) are the same as those used for Braidwood Station, although the subsurface conditions are different. Braidwood subsurface conditions consist of a deep soil profile where as the soil depth to rock at Byron is shallow. The design was performed for the governing conditions.

Lateral design pressures for the river screen house and the essential service water cooling towers subsurface walls were calculated using the procedures given in Subsections 2.5.4.10.5.1 and 2.5.4.10.5.2.

2.5.4.10.5.1 Static Lateral Pressure

The total static lateral pressure was obtained by combining soil and hydrostatic pressures. Static lateral earth pressure on the wall at a depth, h, below grade is given as:

$$Ps = K_o h \ \gamma \qquad (2.5\text{-}3)$$

where:

P_s = Static lateral soil pressure, in psf/linear ft.

K_o = lateral earth pressure coefficient for granular soil compacted against unyielding rigid walls.

h = depth below grade, ft.

γ = soil unit weight, 122 pcf above the water table and 67.6 pcf, submerged unit weight, below the water table

Hydrostatic pressures are calculated using the equation:

$$P_w = 62.4y \qquad (2.5\text{-}4)$$

where:

P_w = hydrostatic pressure, in psf/linear feet.

y = depth below the design water elevation, in feet

The value of the lateral earth pressure coefficient used in design for compacted granular soils behind rigid walls of all Category I structures, except the river screen house, is 0.88. The value used for the river screen house is 0.80.

The above values were determined from analytical procedures using values of the angle of internal friction for compacted granular soils. The values used were 38 degrees for the river screen house and 34 degrees for other Category I structures. There were no lateral earth pressure measurements made.

The plant design water elevation is plant grade, elevation 869 feet. Hydrostatic loading was not a factor in the design of subsurface walls for the essential service water cooling tower because the water table is below the base of the foundation.

2.5.4.10.5.2 Incremental Dynamic Lateral Pressure

The total incremental dynamic lateral pressure was obtained by combining incremental soil and incremental water pressures. The total incremental dynamic lateral pressure was added to the total static lateral pressure to obtain the design lateral pressure.

The dynamic lateral earth pressure increment on the subsurface walls was obtained by methods similar to those developed by Mononabe (Reference 80) and Okabe (Reference 81) and modified by Seed and Whitman (Reference 82).

The equation used to obtain the increment of dynamic lateral earth force was:

$$\Delta P_{AE} - 1/2\ \gamma\ H^2\ \Delta K_{AE} \qquad (2.5\text{-}5)$$

where:

ΔP_{AE} = increment of dynamic lateral earth force in pounds/unit width of wall,

γ = unit weight of soil in pcf; the submerged unit weight was used below the water table,

H = height of the wall in feet,

ΔK_{AE} = dynamic increment in earth pressure coefficient.

Values of ΔK_{AE} were obtained by methods described by Seed and Whitman (Reference 82).

The dynamic earth pressures were assumed to have an inverted triangular distribution, with the resultant acting at two-thirds the height of the wall above the base.

The dynamic pressure increment due to water below the water table, was calculated using the Westergaard theory (Reference 83, modified by Matuo and Ohara, Reference 84). The increase in the pressure on the walls at any depth, y, below the water table is given as:

$$\Delta P_w = 0.70\ CK_h\ (H_1 y)^{1/2} \qquad (2.5\text{-}6)$$

where:

ΔP_w = water pressure, in pounds per feet per unit width of wall,

K_h = $\dfrac{\text{horizontal earthquake ground acceleration}}{g}$

C = $\dfrac{51}{[1.0 - 0.72\,(H_1/1000t)^2]^{1/2}}$ in pcf

t = earthquake period in seconds

H_1 = height of the water table from the base of the wall, in feet

y = depth below the water table, in feet.

2.5.4.11 Design Criteria

The criteria and methods used in the design of Seismic Category I structures are discussed in the following subsections:

a. bearing capacity-Subsection 2.5.4.10,

b. settlement analyses - Subsection 2.5.4.10,

c. slope stability - Subsection 2.5.5, and

d. lateral pressure, Subsection 2.5.4.10.

2.5.4.12 Techniques to Improve Subsurface Conditions

Subsurface conditions at the site were improved by: (1) removal and replacement of overburden materials, and (2) grouting of foundation bedrock.

Information regarding the excavation, removal, and replacement of overburden material is discussed in Subsection 2.5.4.5.

The foundation bedrock supporting the plant Seismic Category I structures and the essential service water cooling towers was grouted by Continental Drilling Company. The results and evaluation of grouting are presented in FSAR Attachments 2.5A and 2.5B.

2.5.4.13 Subsurface Instrumentation

All the Seismic Category I structures, except the river screen house and a portion of the essential service water pipeline, are founded on bedrock. For structures founded on rock, instrumentation of heave and settlement was not warranted because of the small magnitude of elastic settlement discussed in Subsection 2.5.4.10.2.3 and rebound discussed in Subsection 2.5.4.5.1.2.

The river screen house is founded on alluvial soil. Settlement monuments were established in August and December 1977 for the purpose of monitoring settlement of the river screen house. The locations of the settlement monuments are shown on Figure 2.5-247. The settlement records for the river screen house are presented in Table 2.5-31. The settlement readings taken to date indicate that settlement of the river screen house is complete.

The essential service water pipeline is founded on weathered bedrock or soil. Since the weight of the pipeline is less than the weight of the excavated soil, settlement monuments were not established.

2.5.4.14 Construction Notes

With the exception of the circulating water pipeline, no unanticipated conditions were encountered during the construction of the plant facilities which required any change in the design or special construction techniques. For the circulating water pipeline, excessive rainfall resulted in erosion of the granular bedding for the pipeline. The bedding was redesigned such that the pipeline was encased in bash rather than granular fill. During excavation for the main plant, faults of nontectonic nature were observed and the discussion of these faults is presented in FSAR Attachment 2.5C. These findings were included in the Byron PSAR as Amendment 15. After the rock excavation was complete, the design change in the radwaste building area raised the foundation elevation by 20 feet. The over-excavation was backfilled with controlled compacted crushed rock as shown on Figure 2.5-58, Section 6-6, to bring the subgrade to the foundation elevation.

2.5.5 Slope Stability

2.5.5.1 Slope Characteristics

2.5.5.1.1 Plant Site

The Byron Station is located in the gently rolling upland area of the site. The Seismic Category I structures for the main plant are founded on the Dunleith Formation with nominal plant grade established at elevation 869.0 feet (USGS Datum). There are no steep natural slopes present that would be detrimental during the occurrence of a safe shutdown earthquake in the immediate plant area. All artificial slopes in the plant site are less than 10 feet in height, no steeper than 3 horizontal to 1 vertical and will present no safety problems.

2.5.5.1.2 River Screen House

The river screen house is located on the eastern floodplain of the Rock River as shown on Figure 2.5-5. Section DD' on Figure 2.5-60 shows the soils that underlie the river screen house.

The river bank in the vicinity of the river screen house was regraded to a 5 horizontal to 1 vertical slope or flatter. The river screen house intake channel was constructed with side slopes of 5 horizontal to 1 vertical. The slopes of the river screen house intake channel and river bank slope within 50 feet of the river screen house are protected with 3 feet 9 inches of riprap and 9 inches of riprap bedding. An additional 50 feet length of river bank slope on both sides of the river screen house is protected with 18 inches of riprap over 9 inches of riprap bedding.

Relief on the flood plain between the river bank and the bluff line is no more than 5 feet per mile and slope instability is not considered a problem for these slopes. The river screen house is located over 1000 feet from the nearest natural bluff; a distance sufficient so that any unlikely slides need not be considered in the analysis of the structure.

2.5.5.1.3 Essential Service Cooling Tower Makeup Line

The essential service cooling tower makeup line was constructed from the river screen house, station 0+00, to the essential service water cooling tower, station 158+00. Topographic relief along the route is shown on Figure 2.5-6. A geologic cross section along the pipeline route is presented on Figure 2.5-61.

The route traverses the Rock River flood plain from the river screen house, station 0+00, to the base of the bluff, station 10+30, which is approximately 1000 feet east of the Rock River. The ground surface elevation ranges from elevation 680 feet at the river screen house to 700 feet at the bluff-line.

From the base of the bluff, station 10+30, to the crest of the bluff, station 20+10, the makeup line traverses moderately to steeply sloping uplands. The elevation rises from elevation 700 feet at the base of the bluff to 792 feet at the crest of the bluff. The steepest natural slopes along this section of the sloping uplands is 9 horizontal to 1 vertical, which is the steepest natural slope encountered along the essential service cooling tower makeup line.

From the crest of the bluff, station 20+10, to the essential service water cooling towers, station 158+00, the essential service water makeup line traverses level to gently rolling uplands. The elevation rises from elevation 792 feet at the crest of the bluff to elevation 875 feet at the essential service water cooling tower.

The makeup line route is underlain by alluvial deposits from the river screen house to the base of the bluff. The alluvial deposits along the makeup line route are the same as those encountered at the river screen house.

From the bluff to the main plant there is approximately 2.0 feet to 22.0 feet of soil overlying the bedrock. The thickness of dolomitic bedrock overlying the St. Peter Sandstone ranges from approximately 20 feet at the base of the bluff to 220 feet in the plant area. All of the dolomitic bedrock units are nearly horizontally bedded and have the same geologic and engineering characteristics as those encountered at the plant site.

Estimated settlements induced by the fill and backfill in Areas of Concern Nos. 11 and 12 range from 0.25 to 0.75 inch in Area of Concern No. 11 based on the variations in soil thickness and are on the order of 0.5 inch in Area of Concern No. 12.

The settlements were calculated based on the tangent modulus concept as proposed by Janbu (1967). The subsurface profile underlying the pipeline was divided into layers ranging in thickness from 2-10 feet depending on SPT blow count and depth. Each layer was then assigned a modulus number according to its consistency as defined by SPT blow count within the layers. For loose sands (SPT blow count less than 10), a modulus number of 100 was used. For medium dense to dense sands, a modulus number of 200 was used. These numbers are close to lower bound compression moduli for the consistency ranges encountered. NO stress reduction with depth from the backfill load was used. It should be noted that the backfill load in Area of Concern No. 12 represents recompression, since fill was placed in this area prior to excavation of the pipeline trench.

An additional geotechnical investigation consisting of drilling and sampling four soil borings and laboratory testing has been performed to update and confirm the settlement evaluation discussed above. The results of this investigation and subsequent settlement evaluation are presented in the Dames and Moore report entitled, "Report Confirmatory Geotechnical Investigations ESW Pipeline Corridor Byron Station Units 1 and 2 Commonwealth Edison," transmitted by letter from T. R. Tramm to H. R. Denton, dated October 27, 1982. The results given in the report are in agreement with those presented above.

Interpretation of aerial photographs, topographic maps, and field reconnaissance investigations of the bluff area indicate no evidence of any previous slides along the route.

The river bluff is approximately parallel to the regional northeast joint pattern. There is no evidence that this relationship has had an adverse effect on the stability of the natural slopes.

Based on the observations, all slopes will be stable during the occurrence of the safe shutdown earthquake, and thus there would not be any slope failures that would adversely affect the makeup line.

2.5.5.2 Design Criteria and Analysis

2.5.5.2.1 River Screen House Slopes

The results of the slope stability analysis on the river screen house intake channel slopes and river bank slopes indicates that they will remain stable under all the possible combinations of design loads. The most severe loading was postulated by considering rapid drawdown and SSE acting simultaneously. The minimum factor of safety under such loading is greater than 1.1 for all rotation sliding type failures (Reference 85) deeper than 4 feet. This factor of safety would increase if stabilizing effects of riprap were considered. The soil profile and properties used in this analysis are presented in Figure 2.5-102a.

The stability of slopes within the top 4 feet was evaluated using Infinite Slope Analysis (Reference 86). The minimum factor of safety under rapid drawdown and SSE acting simultaneously is greater than 1.10.

2.5.5.3 Logs of Borings

The logs of borings drilled at the location of the river screen house are discussed in Subsection 2.5.4.3.2.3. The logs of borings for the essential service water cooling makeup line are discussed in Subsection 2.5.4.3.2.4.

2.5.5.4 Compacted Fill

All man-made safety-related slopes were excavated into the natural soil and no slopes were constructed using compacted fill.

2.5.6 Embarkments and Dams

There are no man-made embankments or dams at the Byron site.

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TABLE 2.5-1

SUMMARY OF MAJOR FOLDS WITHIN 200 MILES OF THE SITE

NAME	MEANS OF IDENTIFICATION*	MAJOR MOVEMENT**
Ashton Arch	B	Late Paleozoic (Templeton and Willman, Ref. 10)
Baraboo Syncline	S	Precambrian***
(Dalziel and Dott, Ref. 87)		
Brookville Dome	B	Late Paleozoic (Kolata and Buschbach, Ref. 7)
Downs Anticline	B	Late Paleozoic (Cohee and Carter, Ref. 88)
Fond du Lac Syncline	B	Late Paleozoic
Forreston Dome	S, B	Late Paleozoic (Kolata and Buschbach, Ref. 7)
Herscher Dome	B	Late Paleozoic (Templeton and Willman, Ref. 10)
Illinois Basin	S, B, G	Early to late Paleozoic (Eardley, Ref. 21)
Kankakee Arch	S, B, G	Ordovician or Devonian to late Mississippian (Eardley, Ref. 21)
LaSalle Anticlinal Belt	S, B, G	Late Mississippian and Pennsylvanian (Eardley, Ref. 21)

* S = Surface Mapping, B = Borehole, G = Geophysical.
** Final movement considered to be post-Pennsylvanian-pre-Cretaceous.
*** Final movement considered to be Precambrian.

NOTE: Structures listed in this table are shown on Figures 2.5-12, 2.5-14, and 2.5-15.

TABLE 2.5-1 (Cont'd)

NAME	MEANS OF IDENTIFICATION*	MAJOR MOVEMENT**
Leaf River Anticline	S, B	Late Paleozoic (Kotala and Buschbach, Ref. 7)
Lincoln Anticline	S	Late Paleozoic (McCracken, Ref. 89)
Louden Anticline	B	Late Paleozoic (Cohee and Carter, Ref. 88)
Marshall Syncline	B	Late Paleozoic (Clegg, Ref. 9)
Mattoon Anticline	B	Late Paleozoic (Cohee and Carter, Ref. 88)
Meekers Grove Anticline	B	Late Paleozoic (Heyl, et. al., Ref. 19)
Mineral Point Anticline	B	Late Paleozoic (Heyl, et. al., Ref. 19)
Mississippi River Arch	S, B, G	Late Mississippian (Illinois Petroleum 96, Ref. 31)
Murdock Syncline	B	Late Paleozoic (Clegg, Ref. 9)
Oregon Anticline	S, B	Late Paleozoic (Templeton and Willman, Ref. 10)
Pittsfield Anticline	B	Late Paleozoic (Cohee and Carter, Ref. 88)
Polo Basin	B	Late Paleozoic (Templeton and Willman, Ref. 10)

* S = Surface Mapping, B = Borehole, G = Geophysical.

** Final movement considered to be post-Pennsylvanian-pre-Cretaceous.

NOTE: Structures listed in this table are shown on Figures 2.5-12, 2.5-14, and 2.5-15.

TABLE 2.5-1 (Cont'd)

NAME	MEANS OF IDENTIFICATION*	MAJOR MOVEMENT**
Salem Anticline	B	Late Paleozoic (Cohee and Carter, Ref. 88)
Tuscola Anticline	B	Late Paleozoic (Clegg, Ref. 9)
Uptons Cave Syncline	S, B	Late Paleozoic (Kolata and Buschbach, Ref. 7)
Waterloo Syncline	B	Late Paleozoic
Wisconsin Arch	S, B, G	Early to late Paleozoic (Eardley, Ref. 21)

* S = Surface Mapping, B = Borehole, G = Geophysical.

** Final movement considered to be post-Pennsylvanian-pre-Cretaceous.

NOTE: Structures listed in this table are shown on Figures 2.5-12, 2.5-14, and 2.5-15.

TABLE 2.5-2

SUMMARY OF REGIONAL FAULTS

FAULT NAME	MEANS OF IDENTIFICATION*	FAULT TYPE AND DISPLACEMENT	LAST MOVEMENT
Sandwich Fault Zone	S, B, G	Main fault NE side down 900 feet (William and Payne, Ref. 90)	Post-Pennsylvanian-Pre-Cretaceous (McGinnis, Ref. 14)
Plum River Fault Zone	S, B, G (Kolata and Buschbach, Ref. 7)	South side up 100 feet to 400 feet (Kolata and Busch-bach, Ref. 7)	Post-Pennsylvanian (Kolata and Busch-bach, Ref. 7)
Oglesby (Inferred)**	B (Green, Ref. 12)	Down on west side 1200 feet (Green, Ref. 12)	Pre-Cretaceous
Tuscola (Inferred)**	B (Green, Ref. 12)	Down on west side 2000 feet (Green, Ref. 12)	Pre-Cretaceous
Chicago Area Minor Faults (Inferred)	G (Buschbach and Heim, Ref. 15)	Both north and south downthrown blocks, displacement up to	Post-Silurian-Pre-Pleistocene (Buschbach and Heim, Rev. 15)

*S = Surface, B = Borehole, G = Geophysical
**Recent authorities doubt the existence of these faults (References 17 and 13)

Note: Names in parentheses assigned by Dames & Moore. Structures listed in this table are shown on Figure 2.5-13.

TABLE 2.5-2 (Cont'd)

FAULT NAME	MEANS OF IDENTIFICATION*	FAULT TYPE AND DISPLACEMENT	LAST MOVEMENT
		50 feet (Buschbach and Heim, Ref. 15)	
Chicago Area Basement Fault Zone (Inferred)	G, (McGinnis, Ref. 14)	South side down (McGinnis, Ref. 14)	Precambrian (McGinnis, Ref. 14)
Madison (Inferred)**	B (Thwaites, Ref. 16)	North side down (Thwaites, Ref. 16)	Post-Silurian-Pre-Pleistocene (Ostrom, Ref. 17)
Janesville (Inferred)**	B (Thwaites, Ref. 16)	North side down (Thwaites, Ref. 16)	Post-Silurian-Pre-Pleistocene (Ostrom, Ref. 17)
Waukesha (Inferred)**	S (Thwaites, Ref. 91)	Downthrown on south-east side 45 feet (Thwaites, Ref. 13) (Extent of the fault is inferred, Ostrom, Ref. 17)	Post-Siluria-Pre-Pleistocene (Ostrom, Ref. 17)

*S = Surface, B, = Borehole, G = Geophysical
**Recent authorities doubt the existence of these faults (References 17 and 13)

Note: Names in parentheses assigned by Dames & Moore. Structures listed in this table are shown on Figure 2.5-13.

TABLE 2.5-2 (Cont'd)

FAULT NAME	MEANS OF IDENTIFICATION*	FAULT TYPE AND DISPLACEMENT	LAST MOVEMENT
Appleton (Inferred)**	S, B (Thwaites, Ref. 16)	South side down	Post-Silurian-Pre-Pleistocene, Ostrom, Ref. 17)
Green Bay (Inferred)**	S (Thwaites, Ref. 16)	South side down	Post-Silurian-Pre-Pleistocene, Ostrom, Ref. 17)
Mifflin	S. (Heyl, Ref. 19)	Southwest side down 65 feet - 1000 feet strike-slip displacement. (Heyl, Ref. 19)	Pre-Pleistocene (Ostrom, Ref. 17)
Royal Center	B (Becker Ref. 20)	Southeast side down 100 feet (Becker, Ref. 20)	Post-Devonian-Pre-Pleistocene (Gray, Ref. 92)

*S = Surface, B, = Borehole, G = Geophysical
**Recent authorities doubt the existence of these faults (References 17 and 13)

Note: Names in parentheses assigned by Dames & Moore. Structures listed in this table are shown on Figure 2.5-13.

TABLE 2.5-2 (Cont'd)

FAULT NAME	MEANS OF IDENTIFICATION*	FAULT TYPE AND DISPLACEMENT	LAST MOVEMENT
Rough Creek Fault Zone	S (Sutton, Ref. 25) B (Sutton, Ref. 25) G (Woolard and Joesting, Ref. 35)	North side down (Illinois Petroleum 95 and 96, Ref. 30 & 31 (Some members show opposite displacement)	Post-Pennsylvanian-Pre-Pleistocene, possibly Pre-late Cretaceous (Buschbach, Ref. 11)
Northeast Trending Faults North of Rough Creek Fault Zone (Wabash Valley Fault Zone)	S	Both east and west blocks downthrown (Illinois Petroleum 95 and 96, Ref. 30 and 31)	Post-Pennsylvanian-Pre-Pleistocene (Buschbach, Ref.11)
Northeast Trending Faults South of Rough Creek Fault Zone	S, B	Both east and west blocks downthrown (Illinois Petroleum 95 and 96, Ref. 30 and 31)	Post-Pennsylvanian-Pre-Pleistocene (Willman, et. al., Ref. 3)
Ste. Genevieve Fault Zone	S (Sutton, Ref. 25) B (Sutton, Ref. 25) G (Woolard and Joesting, Ref. 35)	North side down 1000 to 2000 feet (Buschbach, Ref. 11)	Post-Pennsylvania-Pre-Pleistocene (Willman, et. al., Ref. 3)

*S = Surface, B, = Borehole, G = Geophysical
**Recent authorities doubt the existence of these faults (References 17 and 13)

Note: Names in parentheses assigned by Dames & Moore. Structures listed in this table are shown on Figure 2.5-13.

TABLE 2.5-3

MINOR FAULTS WITHIN 12 MILES OF THE SITE

NAME	LOCATION	TYPE OF FAULT**	RELATIVE DISPLACE-MENT SIDE DOWN	AMOUNT OF DISPLACE-MENT (feet)	STRIKE	DIP	SHEAR ZONE	DISTANCE FROM PLANT SITE (miles)	AGE LAST MOVEMENT	COMMENTS
1. Oregon*	Sec. 3 and 4, T.23N., R.10E	N	SW	235-515	WNW			4.5	Ordovician	Detected by borehole data
2.	NE 1/4, NE 1/4, SE 1/4, Sec. 30, T.25N., R.11E.	N	N	Net 15 in.	NW	Vertical	35 feet, wide, 6 feet wide breccia zone 5 miniature faults	3.5	Post-Pennsylvanian, pre-Cretaceous	5 miniature faults and 2 breccia zones
3.	SW 1/4, SE 1/4, NW 1/4, Sec. 2, T.23N., R.10E	N	S	5	N.60° W. to N. 72° W.	Vertical to 75°	Shattering to complete brecciation	4.2	Post-Pennsylvanian, pre-Cretaceous	
4.	SE 1/4, SW 1/4, NE 1/4, Sec. 2, T.23N., R.10E	N	N	25	N.67° W.	Vertical	3 feet wide	4.2	Post-Pennsylvanian, pre-Cretaceous	
5.	SE 1/4, SW 1/4, NE 1/4, Sec. 2, T.23N., R.10F	S			N.40° E.	Vertical	2 breccia zones 20 feet wide, 30 feet apart	4.2	Post-Pennsylvanian, pre-Cretaceous	

TABLE 2.5-3 (Cont'd)

NAME	LOCATION	TYPE OF FAULT**	RELATIVE DISPLACEMENT SIDE DOWN	AMOUNT OF DISPLACEMENT (feet)	STRIKE	DIP	SHEAR ZONE	DISTANCE FROM PLANT SITE (miles)	AGE LAST MOVEMENT	COMMENTS
6.	NE 1/4, NW 1/4, SE 1/4, Sec. 2, T.23N., R.10E.	N	N	35	N.57° E.	70° NW	1 and 1/2-foot zone, strongly sheared	4.6	Post-Pennsylvanian, pre-Cretaceous	
7.	N 1/2, NE 1/4, NW 1/4, SE 1/4, Sec. 1, T.23N, R.10E.	N	SE	15	N.65° E.	Vertical		4	Post-Pennsylvanian, pre-Cretaceous	
8. Oregon Thrust Fault*	N 1/2, Sec. 3, T.23N., R.10E.	T	NE		N.35° W.		St. Peter altered to quartzite breccia	4.3	Post-Pennsylvanian, pre-Cretaceous	
9. Stronghold	NW 1/4, Sec. 33, T.24N., R.10E.	N	SW	120	N.59° W.			3.8	Post-Pennsylvanian, pre-Cretaceous	
10.	SE 1/4, SE 1/4, NW 1/4, and NE 1/4, NE 1/4, SW 1/4, Sec. 33, T.24N., R.10E.	N	SW	24	N.48° W.			3.9	Post-Pennsylvanian, pre-Cretaceous	
11.	NE 1/4, NW 1/4, NE 1/4, Sec. 32, T.24N., R.10E	N	SW	<35	N.55° W	>35° SW	20-foot wide sheared brecciated zone	4.9	Post-Pennsylvanian, pre-Cretaceous	

TABLE 2.5-3 (Cont'd)

NAME	LOCATION	TYPE OF FAULT**	RELATIVE DISPLACEMENT SIDE DOWN	AMOUNT OF DISPLACEMENT (feet)	STRIKE	DIP	SHEAR ZONE	DISTANCE FROM PLANT SITE (miles)	AGE LAST MOVEMENT	COMMENTS
12.	Center of West Line SE 1/4, SE 1/4, Sec. 29, T.24N., R.10E	N	SW	>14 <31	NW			4.3	Post-Pennsylvanian, pre-Cretaceous	
13.	SE 1/4, NE 1/4, SE 1/4, Sec. 29, T.24N., R.10E.	N	SW	<18	N.40° W.	Vertical	Narrow shear breccia zone	4.4	Post-Pennsylvanian, pre-Cretaceous	Shears 20 feet away N.65° W. dip. 60°-80° NNE
14.	NE 1/4, NE 1/4, SW 1/4, Sec. 28, T.24N., R.10E.		Net N 2 Feet	≤2	5 N.58°-75°W	60°SW - 60°NE	Brecciated with weathered clay gouge	3.8	Post-Pennsylvanian, pre-Cretaceous	Horst and graben mosaic fault zone with 7 faults 75 feet wide
					Z N.10°E	Vertical				
15. Mt. Morris* Disturbance	S 1/2, Sec. 25, T.24N., R.10E.				ENE to WNW	Essentially vertical	23 breccia zones or faults 1-120 feet; average 10 feet	6.5	Post-Pennsylvanian, pre-Cretaceous	
16. Mt. Morris*	Sec. 21-22, 27-28, T.24N., R.10E.	N	NW	158	N.50° E.			8	Post-Pennsylvanian, pre-Cretaceous	Fault zone covered

TABLE 2.5-3 (Cont'd)

NAME	LOCATION	TYPE OF FAULT**	RELATIVE DISPLACE-MENT SIDE DOWN	AMOUNT OF DISPLACE-MENT (feet)	STRIKE	DIP	SHEAR ZONE	DISTANCE FROM PLANT SITE (miles)	AGE LAST MOVEMENT	COMMENTS
17.	NE 1/4, NW 1/4, SW 1/4, Sec. 6, T.24N., R.8E.	N	S	40-60	N.55° W.	35°W	Minor Shears	10	Post-Pennsylvanian, pre-Cretaceous	
18. Adeline Thrust Fault*	SW 1/4, NW 1/4, NE 1/4, Sec. 32, T.25N., R.9E.	T	S	Un-known <35	N.75° E.	16°SSE		10.5	Post-Pennsylvanian, pre-Cretaceous	
19.	SE 1/4, SE 1/4, SE 1/4, Sec. 19, T.25N., R.9E.	N	E	23	N.60° W.			11.5	Post-Pennsylvanian, pre-Cretaceous	Small graben between 19 and 20 feet with at least 9 minor normal faults with a maximum of 2 feet total displacement
20.	SW 1/4, SW 1/4, SE 1/4, Sec. 20, T.25N., R.9E.		SW	21	N.60° W.		10-foot breccia and shear zone	11	Post-Pennsylvanian, pre-Cretaceous	

TABLE 2.5-3 (Cont'd)

NAME	LOCATION	TYPE OF FAULT**	RELATIVE DISPLACE-MENT SIDE DOWN	AMOUNT OF DISPLACE-MENT (feet)	STRIKE	DIP	SHEAR ZONE	DISTANCE FROM PLANT SITE (miles)	AGE LAST MOVEMENT	COMMENTS
21.	SW 1/4, SW 1/4, SE 1/4, Sec. 20, T.25N., R.9E.		53					11.5	Post-Pennsylvanian, pre-Cretaceous	Fault?
22.	NE 1/4, SE 1/4, SE 1/4, Sec. 20, T.25N., R.9E.		W	39	NW			11	Post-Pennsylvanian, pre-Cretaceous	Fault?
23.	SE 1/4, SW 1/4, SW 1/4, Sec. 20, T.25N., R.9E.	N	W	46	NW		Not visible	11	Post-Pennsylvanian, pre-Cretaceous	
24.	NW 1/4, SW 1/4, SW 1/4, Sec. 20, T.25N., R.9E.	N	NE	12	N.60° W.	Vertical	3-foot wide breccia zone	10.5	Post-Pennsylvanian, pre-Cretaceous	Horst between 23 and 24
25.	NE 1/4, SE 1/4, SE 1/4, Sec. 20, T.25N., R.9E.	N	NW	4 in.	N.37° E.	Vertical	1-foot intense shearing and breccia	10.5	Post-Pennsylvanian, pre-Cretaceous	
26.	NW 1/4, SW 1/4, SW 1/4, Sec. 21, T.25N., R.9E.		N		N.75° E.		5-foot wide breccia zone	10.5	Post-Pennsylvanian, pre-Cretaceous	

TABLE 2.5-3 (Cont'd)

NAME	LOCATION	TYPE OF FAULT**	RELATIVE DISPLACE-MENT SIDE DOWN	AMOUNT OF DISPLACE-MENT (feet)	STRIKE	DIP	SHEAR ZONE	DISTANCE FROM PLANT SITE (miles)	AGE LAST MOVEMENT	COMMENTS
27.	SE 1/4, SW 1/4, SW 1/4, Sec. 21, T.25N., R.9E.				N.67° W.		25-foot wide intense breccia zone	10.5	Post-Pennsylvanian, pre-Cretaceous	May represent thrust fault
28.	SE 1/4, SE 1/4, SW 1/4, Sec. 21, T.25N., R.9E.				Avg. N.67° W.	Vertical	Shear zone with 2 foot breccia zone	10.5	Post-Pennsylvanian, pre-Cretaceous	
					N.60° E - N.75° E.	Vertical to SW				
29.	NE 1/4, NE 1/4, NW 1/4, Sec. 21, T.25N., R.9E			2.3 17			Shatter zone	10	Post-Pennsylvanian, pre-Cretaceous	3 small faults
30.	SE 1/4, SE 1/4, SW 1/4, Sec. 21, T.25N., R.9E.	N	S	12	N.45° W.	Vertical zone	Curved shears	10	Post-Pennsylvanian, pre-Cretaceous	Fault?
31. White Eagle Thrust Zone**	SW 1/4, SW 1/4, SE 1/4, Sec. 21, T.25N., R.9E.	T	NE	14	N.45° W.	55° NE	2 and 1/2-foot shear brecciated powdered rock	10	Post-Pennsylvanian, pre-Cretaceous	

TABLE 2.5-3 (Cont'd)

NAME	LOCATION	TYPE OF FAULT**	RELATIVE DISPLACE-MENT SIDE DOWN	AMOUNT OF DISPLACE-MENT (feet)	STRIKE	DIP	SHEAR ZONE	DISTANCE FROM PLANT SITE (miles)	AGE LAST MOVEMENT	COMMENTS
32. Mud Creek Thrust Fault	NW 1/4, SE 1/4, Sec. 15, T.25N., R.9E.	T	SW		N.55° W.	70° NE	40-foot wide breccia rotation and shear-ing zone	10.5	Post-Pennsylvanian, pre-Cretaceous	
33.	SE 1/4, SW 1/4, Sec. 15, T.25N., R.9E.				NE	NW	Closely spaced shears and joints with shattering and breccia	10	Post-Pennsylvanian, pre-Cretaceous	
34.	NW 1/4, SW 1/4, NE 1/4, Sec. 22, T.25N., R.9E.						3 breccia zones, 2 feet wide	9.5	Post-Pennsylvanian, pre-Cretaceous	
35.	SE 1/4, NW 1/4, SE 1/4, Sec. 25, T.25N., R.9E.			33			Slicks in area	7.4	Post-Pennsylvanian, pre-Cretaceous	
36.	SE 1/4, SE 1/4, SW 1/4, Sec. 17, T.25N., R.10E.			36				7	Post-Pennsylvanian, pre-Cretaceous	Fault zone covered
37.	NW Sec. 5, and NW Sec. 6, T.23N., R.10E.	N	N	30-40	N.67° W.	Steep NNE		6.0	Post-Pennsylvanian, pre-Cretaceous	Family of faults, sharp up-drag

TABLE 2.5-3 (Cont'd)

NAME	LOCATION	TYPE OF FAULT**	RELATIVE DISPLACE-MENT SIDE DOWN	AMOUNT OF DISPLACE-MENT (feet)	STRIKE	DIP	SHEAR ZONE	DISTANCE FROM PLANT SITE (miles)	AGE LAST MOVEMENT	COMMENTS
38.	SE 1/4, SW 1/4, NW 1/4, Sec. 35, T.26N., R.9E.		N S N S N	6" 2" 13" 6" 11"	N.84° W. E-W N.87° W.	83°S. Vertical " " 81°N.	Thin breccia zones (inch) slicks	11.5	Post-Pennsylvanian, pre-Cretaceous	5 very small faults, horst and graben
			Net North	1'-10"	N.69° W E-W					
39. Mud Creek Fault*	NW 1/4, SW 1/4, SE 1/4, Sec. 30, T.24N., R.10E.	N	N	100	N.70° W.	Steep north to vertical	25-foot breccia zone	5.5	Post-Pennsylvanian, pre-Cretaceous	
40.	SE 1/4, NE 1/4, Sec. 16, T.23N., R.9E.	N	W	2.5	N.25° W.	70°W		11	Post-Pennsylvanian, pre-Cretaceous	
41. FSGI No. 3	S 1/2, SE 1/4, Sec. 23, T.24N., R.10E.	N	E	1" to 2"	N.3° W.	N.85° E.		0.0	Post-Pennsylvanian, pre-Cretaceous	Described in Attach-ment 2.5C to Chapter 2.0
42. FSGI No. 4	S 1/2, SE 1/4, Sec. 23 T.24N., R.10E.	N	E	1"	N.22° W.	N.82° E.		0.0	Post-Pennsylvanian, pre-Cretaceous	Described in Attach-ment 2.5C to Chapter 2.0

TABLE 2.5-3 (Cont'd)

NAME	LOCATION	TYPE OF FAULT**	RELATIVE DISPLACE-MENT SIDE DOWN	AMOUNT OF DISPLACE-MENT (feet)	STRIKE	DIP	SHEAR ZONE	DISTANCE FROM PLANT SITE (miles)	AGE LAST MOVEMENT	COMMENTS
43. FSGI No. 10	S 1/2, SE 1/4, Sec. 23, T.24N., R.10E.	N	S	0.5"	N.76° W.	Variable high angle		0.0	Post-Pennsylvanian, pre-Cretaceous	Described in Attach-ment 2.5C to Chapter 2.0
44. FSGI No. 11	S 1/2, SE 1/4, Sec. 23, T.24N., R.10E.	N	S	1/2"	N.46° W.	High angle		0.0	Post-Pennsylvanian, pre-Cretaceous	Described in Attach-ment 2.5C to Chapter 2.0
45. FSGI No. 12	S 1/2, SE 1/4, Sec. 23, T.24N., R.10E.	N	N	1/2" to 1"	N.67° W.	Vertical		0.0	Post-Pennsylvanian, pre-Cretaceous	Described in Attach-ment 2.5C to Chapter 2.0
46. FSGI No. 13	S 1/2, SE 1/4, Sec. 23, T.24N., R.10E.	N	S	1"	N.81° W.	Variable high angle		0.0	Post-Pennsylvanian, pre-Cretaceous	Described in Attach-ment 2.5C to Chapter 2.0
47. FSGI No. 22	S 1/2, SE 1/4, Sec. 23, T.24N., R.10E.	N	S	2" to 3"	N.72° W.	Vertical		0.0	Post-Pennsylvanian, pre-Cretaceous	Described in Attach-ment 2.5C to Chapter 2.0
48. FSGI No. 23	S 1/2, SE 1/4, Sec. 23, T.24N., R.10E.	N	S	0.5"	N.64° W.	High angle		0.0	Post-Pennsylvanian, pre-Cretaceous	Described in Attach-ment 2.5C to Chapter 2.0

TABLE 2.5-3 (Cont'd)

NAME	LOCATION	TYPE OF FAULT**	RELATIVE DISPLACE-MENT SIDE DOWN	AMOUNT OF DISPLACE-MENT (feet)	STRIKE	DIP	SHEAR ZONE	DISTANCE FROM PLANT SITE (miles)	AGE LAST MOVEMENT	COMMENTS
49. FSGI No. 25	S 1/2, SE 1/4, Sec. 23, T.24N., R.10E.	N	S	3"	N.56° W.	Vertical		0.0	Post-Pennsylvanian, pre-Cretaceous	Described in Attach-ment 2.5C to Chapter 2.0
50. FSGI No. 34	S 1/2, SE 1/4, Sec. 23, T.24N., R.10E.	N	S	5"	N.73° W.	High angle		0.0	Post-Pennsylvanian, pre-Cretaceous	Described in Attach-ment 2.5C to Chapter 2.0
51. FSGI No. 39	S 1/2, SE 1/4, Sec. 23 T.24N., R.10E.	N	N	1"	N.77° W.	Vertical		0.0	Post-Pennsylvanian, pre-Cretaceous	Described in Attach-ment 2.5C to Chapter 2.0

* Name applied by Templeton, 1952

** N = Normal Fault, T = Thrust Fault, S = Shear Fault

Note: All faults detected by surface data unless otherwise indicated.
Faults numbered 1 through 40 of this table are shown on Figure 2.5-16.
Faults numbered 41 through 51 are described and shown in Attachment 2C.3

TABLE 2.5-4

ROCK QUALITY DESIGN

RQD	ROCK QUALITY
0 - 25%	Very Poor
25 - 50%	Poor
50 - 75%	Fair
75 - 90%	Good
90 - 100%	Excellent

TABLE 2.5-5

DEGREE OF WEATHERING

TERM	ABBREVIATION	DESCRIPTION
Fresh	Fr	The rock shows no discoloration, loss of strength or any other effect due to weathering.
Slightly Weathered	SW	The rock is slightly discolored, but not noticeably lower in strength than the fresh rock.
Moderately Weathered	MW	The rock is discolored and noticeably weakened, but 2-inch diameter drill cores cannot usually be broken up by hand, across the rock fabric.
Highly Weathered	HW	The rock is usually discolored and weakened to such an extent that 2-inch diameter cores can be broken up readily by hand, across the rock fabric. Wet strength usually much lower than dry strength.

TABLE 2.5-6

MEAN WATER LOSS FOR THE DUNLEITH FORMATION

BORING NUMBER	WATER LOSS UNITS	INTERVAL TESTED (ft)	INTERVAL WHERE WATER LOSSES OCCURRED (ft)	ELEVATION WHERE WATER LOSSES OCCURRED (MSL)
P-2	5.9×10^{-4}	22-105.5	22-105.5	856.3-772.8
P-3	9.1×10^{-4}	22-100.5	62-100.5	811.8-773.3
P-4	5.5×10^{-4}	22-82[1]	22-82	859.6-798.6
P-5	1.8×10^{-3}	23-98	23-98	849.8-774.8
P-6	2.8×10^{-3}	22-100.5	22-100.5	853.6-775.1
P-7	1.8×10^{-3}	21-104	31-104	846.5-773.5
P-9	2.1×10^{-2}	22-521	22-52	839.3-809.3
P-10	4.5×10^{-3}	21.5-87	41.5-87	824.6-779.1
P-15	9.0×10^{-3}	21-96	21-96	855.9-780.9
P-17	1.5×10^{-3}	22-721	22-72	855.2-805.2
P-22	5.7×10^{-3}	22-103	22-103	855.3-774.3
P-23	3.5×10^{-3}	22-94	22-82	852.8-792.84

[1]The entire Dunleith Formation was not tested due to packer refusal.

TABLE 2.5-7

CHEMICAL COMPOSITION OF GROUNDWATER

BORING NUMBER	G-7	P-4	P-5	P-8	P-9	P-10	P-22	P-39
GROUP IN WHICH PIEZOMETER INSTALLED*	G	G	P	A	A	P	G	P
PARAMETERS								
Chloride (ppm)	6.7	5.3	6.0	7.0	<1.0	8.0	5.9	8.5
Sulfate (ppm)	39.0	21.0	34.0	28.0	27.0	21.0	29.0	31.0
Nitrate (ppm)	0.66	----	----	----	0.18	----	----	----
Sodium (ppm)	14.0	----	----	----	6.0	----	----	----
Calcium (ppm)	150.0	----	----	----	110.0	----	----	----
Magnesium (ppm)	50.0	----	----	----	50.0	----	----	----
Aluminum	2.0	----	----	----	<1	----	----	----
Copper (ppm)	0.1	----	----	----	<0.1	----	----	----
Iron (ppm)	8.1	----	----	----	4.4	----	----	----
Lead (ppm)	0.4	----	----	----	0.1	----	----	----
Mercury (ppb)	0.1	----	----	----	0.1	----	----	----
Manganese (ppm)	0.7	----	----	----	5.5	----	----	----
pH	7.92	7.30	7.32	6.86	5.92	7.31	7.13	7.24

* G = Galena
  P = Platteville
  A = Ancell
< Denotes "less than"

TABLE 2.5-8

MODIFIED MERCALLI INTENSITY (DAMAGE) SCALE OF 1931
(Abridged)

I. Not felt except by a very few under especially favorable circumstances. (I, Rossi-Forel Scale)

II. Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing. (I or II, Rossi-Forel Scale)

III. Felt quite noticeably indoors, especially on upper floors of buildings but many people do not recognize it as an earthquake. Standing motorcars may rock slightly. Vibration like passing of truck. Duration estimated. (III, Rossi-Forel Scale)

IV. During the day, felt indoors by many, outdoors by few. At night, some awakened. Dishes, windows, doors disturbed; walls make creaking sound. Sensation like heavy truck striking building. Standing motorcars rock noticeably. (IV to V, Rossi-Forel Scale)

V. Felt by nearly everyone, many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbances of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop. (V to VI, Rossi-Forel Scale)

VI. Felt by all, many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight. (VI to VII, Rossi-Forel Scale)

VII. Everybody runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken. Noticed by persons driving motorcars. (VIII, Rossi-Forel Scale)

VIII. Damage slight in specially designed structures; considerable in ordinary substantial buildings, with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving motorcars disturbed. (VIII+ to IX-, Rossi-Forel Scale)

TABLE 2.5-8 (Cont'd)

IX. Damage considerable in specially designed structures; well designed frame structures thrown out of plumb; great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken. (IX+, Rossi-Forel Scale)

X. Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed (slopped) over banks. (X, Rossi-Forel Scale)

XI. Few, if any (masonry), structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipelines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.

XII. Damage total. Waves seen on ground surface. Lines of sight and level distorted. Objects thrown upward into the air.

TABLE 2.5-9

EARTHQUAKE EPICENTERS

38° TO 46° NORTH LATITUDE

84° TO 94° WEST LONGITUDE

DATE	LOCATION	NORTH LATITUDE (degrees)	WEST LONGITUDE (degrees)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
1804 Aug 24	Fort Dearborn, IL	42.0	87.8	VI-VII	30,000	1,2,3,5
1818 Apr 11	St. Louis, MO	38.6	90.2	III-IV	7,500	1
1819 Sept 16	Randolph County, IL	38.1	89.8	IV	9,600	1
1819 Sept 16	Randolph County, IL	38.1	89.8	III-IV		1
1827 July 5	St. Louis, MO	38.6	90.2	IV-V		1
1827 July 5	Grant County, KY	38.7	84.6	IV	15,000	1,2
1827 July 5	New Albany, IN	38.3	85.8		165,000	1,2
1827 July 6	Cincinnati, OH	39.1	84.5	IV		1
1827 Aug 6	New Albany, IN	38.3	85.8	VI		1,2,3,5
1827 Aug 7	New Albany, IN	38.3	85.8	VI		1,2,3,5
1827 Aug 14	St. Louis, MO	38.6	90.2	III		1
1838 June 9	St. Louis, MO	38.5	90.3	VI	300	1
1843 Feb 16	St. Louis, MO	38.6	90.2	IV-V	100,000	1

TABLE 2.5-9 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (degrees)	WEST LONGITUDE (degrees)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
1845	Putnam County, OH	41.1	84.2	II		1
1850 April 4	Louisville, KY	38.3	85.8	V		1,2,4
1854 Feb 28	Lexington, KY	38.1	84.5	VI		15
1857 Oct 8	St. Louis, MO	38.6	90.3	VI-VII	7,500	1,3
1865	Le Sueur, MN	44.5	93.9	VI-VII		1,2
1869 Feb 20	Lexington, KY	38.1	84.5	III-IV		1
1871 July 25	St. Clair County, IL	38.5	90.0	III	1,000	1
1872 July 8	Chillicothe, MO	39.8	93.6	III		1
1873 April 22	Dayton, OH	39.8	84.2	III-IV		1
1875 June 18	Champaign County, OH	40.2	84.0	VII	40,000	1,2,6,8,14
1876 Jan 27	Adrian, MI	41.9	84.0			1
1876 June	Anna, OH	40.4	84.2			1,8,14
1876 Sept 24	Wabash County, IL	38.5	87.9	VI		1
1876 Sept 25	Knox County, IN	38.5	87.7	VI	60,000	1,2,3,6,7
1876 Sept 26	Wabash County, IL	38.5	87.9	III		1

TABLE 2.5-9 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (degrees)	WEST LONGITUDE (degrees)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
1877 May 26	New Harmony, IN	38.1	87.9	III-IV		1
1881 April 20	Goshen, IN	41.6	85.8	IV		1
1881 May 27	La Salle, IL	41.3	89.1	VI		1,2
1881 Aug 29	Hillsboro, OH	39.2	83.6	III		1
1882 Feb 9	Anna, OH	40.4	84.2	V	100	1,2,3,8,14
1882 July 20	Randolph County, IL	38.0	90.0	V	30,000	1,2
1882 Sept 27	Macoupin County, IL	39.0	90.0	VI	25,000	1,2,3
1882 Oct 14	Macoupin County, IL	39.0	90.0	V	8,000	1,2
1882 Oct 15	Macoupin County, IL	39.0	90.0	V	8,000	1,2,3
1882 Oct 22	Greenville, IL	38.9	89.4	III		1
1882 Nov 15	St. Louis, MO	38.6	90.2	III		1
1883 Feb 4	Kalamazoo County, MI	42.3	85.6	VI	150,000	1,2,3
1883 Nov 14	St. Louis, MO	38.6	90.2	IV	1,200	1
1883 Dec 28	Bloomington, IL	40.5	87.0	III		16
1884 Mar 31	Preble County, OH	39.6	84.8	II		1
1884 Sept 19	Allen County, OH	40.7	84.1	VI	125,000	1,2,8,14

TABLE 2.5-9 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (degrees)	WEST LONGITUDE (degrees)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
1884 Dec 23	Anna, OH	40.4	84.2	III		1,5,14
1885 Dec 26	Bloomington, IL	40.5	89.0	III		1
1886 Mar 1	Butlerville, IN	39.0	85.5	IV		1,2
1886 Aug 13	Indianapolis, IN	39.8	86.2	IV-V		1
1887 Feb 6	Vincennes, IN	38.7	87.4	VI	75,000	1,2,3,6,7
1889 Sept	Anna, OH	40.4	84.2	III		1,8,14
1891 July 26	Evansville, IN	38.0	87.6	VI		1,2,3,6
1892	Anna, OH	40.4	84.2			1,8,14
1896 Mar 15	Sidney, OH	40.3	84.2	IV		1,8,14
1897 Oct 31	Niles, MI	41.8	86.3			1
1899 Feb 8	Chicago, IL	41.9	87.6			1
1899 Feb 9	Chicago, IL	41.9	87.6			1
1899 Apr 29	Dubois County, IN	38.5	87.0	VII	40,000	1,2,6,7,9
1899 Oct 10	St. Joseph, MI	42.1	86.5	IV		1
1899 Oct 12	Kenosha, WI	42.6	87.8			1

TABLE 2.5-9 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (degrees)	WEST LONGITUDE (degrees)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
1902 Jan 24	Maplewood, MO	38.6	90.3	VI	40,000	1,3
1902 Mar 10	Hagerstown, IN	39.9	85.2	III-IV		1
1903 Jan 1	Hagerstown, IN	39.9	85.2	II-III		1
1903 Feb 8	St. Louis, MO	38.6	90.3	VI	40,000	1,3
1903 Mar 17	Hillsboro, IL	39.2	89.5	III-IV		1
1903 Sept 20	Morgantown, IN	39.4	86.3	IV		1
1903 Sept 21	Olney, IL	38.7	88.1	IV		1
1903 Nov 4	St. Louis, MO	38.6	90.3	VI-VII	70,000	1,3
1903 Nov 20	Morgantown, IN	39.4	86.3			1
1903 Dec 11	Effingham, IL	39.1	88.5	II		1
1903 Dec 31	Fairmont, IL	41.6	88.1			1
1905 Mar 31	Menominee, MI	45.0	87.7	V		1,3
1905 Apr 13	Keokuk, IA	40.4	91.6	IV-V	5,000	1,2,3
1905 Aug 22	Quincy, IL	39.9	91.4	II-III		1
1906 Feb 23	Anabel, MO	39.7	92.4	III		1
1906 Mar 6	Hannibal, MO	39.7	91.4	IV		1

TABLE 2.5-9 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (degrees)	WEST LONGITUDE (degrees)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
1906 Apr 22	Milwaukee, WI	43.0	87.9			1
1906 Apr 24	Milwaukee, WI	43.0	87.9			1
1906 May 8	Shelby County, IN	39.5	85.8	III-IV	600	1
1906 May 9	Columbus, IN	39.2	85.9	IV		1,2,3
1906 May 11	Petersburg, IN	38.5	87.3	V	1,200	1,2,3
1906 May 19	Grand Rapids, MI	43.0	85.7			1
1906 May 21	Flora, IL	38.7	88.5	V	580	1,2,3,6
1906 Aug 13	Greencastle, IN	39.6	86.9	IV		1
1906 Sept 7	Owensville, IN	38.3	87.7	IV	500	1
1906 Nov 23	Anabel, MO	39.7	92.4	III		1
1907 Jan 10	Menominee, MI	45.1	87.6			1
1907 Jan 29	Morgan County, IN	39.5	86.6	V		1,2
1907 Jan 30	Greenville, IL	38.9	89.4	V		1
1907 Nov 20	Stephenson County, IL	42.3	89.8	IV	100	1,2
1907 Nov 28	Stephenson County, IL	42.3	89.8	IV	100	1,2

TABLE 2.5-9 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (degrees)	WEST LONGITUDE (degrees)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
1907 Dec 10	St. Louis, MO	38.6	90.2	IV		1
1908 Nov 12	Sedalia, MO	38.7	93.2	IV	700	1
1909 May 26	South Beloit, Wis.	42.5	89.0	VII	170,000	1,2,3,5
1909 July 18	Mason County, IL	40.2	90.0	VII	35,000	1,2,3
1909 Aug 16	Monroe County, IL	38.3	90.2	IV-V	18,000	1
1909 Sept 22	Lawrence County, IN	38.7	86.5	V	4,000	1,2,3
1909 Sept 27	Robinson, IL	39.0	87.7	VII	30,000	1,2,3,6,10
1909 Sept 27	Vincennes, IN	38.7	87.5	V	4,000	1,2,3,6,10
1909 Oct 22	Sterling, IL	41.8	89.7	IV-V		1,2
1909 Oct 22	Near Scott, KY	38.9	84.5			1
1909 Oct 23	Robinson, IL	39.0	87.7	V	14,000	1,2,5
1911 Feb 28	St. Louis County, MO	38.7	90.3	IV		1
1911 July 29	Chicago, IL	41.9	87.6	IV-V		1,2
1912 Jan 2	Kendall County, IL	41.5	88.5	VI	40,000	1,3
1912 Sept 25	Rockford, IL	42.3	89.1	III-IV		1,2
1913 Oct 16	Sterling, IL	41.8	89.7	III-IV	4,000	1,2

TABLE 2.5-9 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (degrees)	WEST LONGITUDE (degrees)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
1913 Nov 11	Louisville, KY	38.3	85.8	IV		1
1914 Oct 7	Madison, WI	43.1	89.4	IV		1
1914	Anna, OH	40.4	84.2	II		1,8,14
1915 Apr 15	Olney, IL	38.7	88.1	II-III	3,000	1
1916 Jan 7	Worthington, IN	39.1	87.0	III	3,000	1
1916 May 31	Madison, WI	43.1	89.4	II		1
1916	Clarke County, IA	41.1	93.8	II-III		1
1917 Apr 9	Jefferson County, MO	38.1	90.6	VI	200,000	1,3
1918 Feb 22	Shiawassee County, MI	42.9	84.2	IV		1
1918 July 1	Hannibal, MO	39.7	91.4	IV		1
1919 May 25	Knox County, IN	38.5	87.5	V	18,000	1,2,3,6
1920 Apr 30	Centralia, IL	38.5	89.1	IV	4,000	1
1920 May 1	St. Louis County, MO	38.5	90.5	V	10,000	1,3
1921 Mar 14	Crawfordsville, IN	40.0	86.9	IV	25,000	1
1921 Sept 8	Waterloo, IL	38.3	90.2	IV	4,000	1

TABLE 2.5-9 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (degrees)	WEST LONGITUDE (degrees)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
1921 Oct 9	Waterloo, IL	38.3	90.2	III	3,000	1
1922 Apr 10	Monmouth, IL	40.9	90.7	II		1
1922 July 7	Fond du Lac, WI	43.8	88.5	V		1,2
1923 Mar 8	Greenville, IL	38.9	89.4	III-IV	4,000	1
1923 Nov 9	Tallula, IL	40.0	89.9	V	600	1,2,3
1925 Jan 26	Waterloo, IA	42.5	92.3	II	200	1
1925 Mar 3	Evanston, IL	42.0	87.7	II-III		1
1925 Apr 4	Cincinnati, OH	39.1	84.5			1,8,14
1925 Apr 26	Vanderburgh County, IN	38.0	87.5	VI	100,000	1,2,3
1925 July 13	Edwardsville, IL	38.8	90.0	V		1
1925 Oct	Anna, OH	40.4	84.2	II		1,8,14
1926 Oct 3	Princeton, IN	38.4	87.6	III		1
1928 Jan 23	Near Mount Carroll, IL	42.0	90.0	IV	400	1,2
1928 Mar 17	St. Louis, MO	38.6	90.2	I		1
1928 Oct 27	Shelby County, OH	40.4	84.1	III	100	1,8,14
1929 Feb 14	Near Princeton, IN	38.3	87.6	III-IV	1,000	1

TABLE 2.5-9 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (degrees)	WEST LONGITUDE (degrees)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
1929 Mar 8	Shelby County, OH	40.4	84.2	V	5,000	1,2,3,6,8,14
1930 May 28	Near Hannibal, MO	39.7	91.3	III		1
1930 Jun 26	Near Lima, OH	40.5	84.0	IV		1,8,14
1930 Jun 27	Near Lima, OH	40.5	84.0	IV		1,8,14
1930 Aug 8	Near Hannibal, MO	39.6	91.4	III-IV		1
1930 Sept 20	Anna, OH	40.4	84.2	VI		1,2,3,8,11,14
1930 Sept 29	Sidney, OH	40.3	84.2	III		1,8,14
1930 Sept 30	Anna, OH	40.3	84.3	VII		1,2,3,8,9,14
1930 Oct	Anna, OH	40.4	84.2	III-IV		1,8,14
1930 Dec 23	Near St. Louis, MO	38.6	90.5	III-IV	1,000	1
1931 Jan 5	Elliston, IN	39.0	86.9	V	500	1,2,3,12
1931 Mar 21	Sidney, OH	40.3	84.2	III		1,8,14
1931 Mar 31	Shelby County, OH	40.4	84.1	III		1
1931 Jun 10	Malinta, OH	41.3	84.0	V	1,500	1,8,14
1931 Sept 20	Anna, OH	40.4	84.2	VII	45,400	1,2,3,8,11,12

TABLE 2.5-9 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (degrees)	WEST LONGITUDE (degrees)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
1931 Oct 8	Anna, OH	40.4	84.2	III		1,8,14
1931 Oct 18	Madison, WI	43.1	89.4	III		1
1931 Dec 17	St. Louis, MO	38.6	90.2	II		1
1931 Dec 31	Petersburg, IN	38.5	87.3			1
1933 Feb 22	Sidney, OH	40.3	84.2	III-IV	2,000	1
1933 Nov 16	Grover, MO	38.6	90.6	III-IV	1,500	1
1933 Dec 6	Stoughton, WI	42.9	89.2	IV	5,000	1,2,3
1934 Nov 12	Rock Island, IL	41.5	90.5	VI	5,000	1,3
1935 Jan 5	Moline, IL	41.5	90.6	IV	200	1,2
1935 Jan 30	Harrison County, MO	40.5	94.0	III		1
1935 Feb 26	Burlington, IA	40.8	91.2	III		1
1935 Oct 29	Pike County, IL	39.6	90.8			1
1936 Oct 8	Butler County, OH	39.3	84.4	III	700	1,8,14
1936 Dec 25	Cincinnati, OH	39.1	84.5	III		1
1937 Mar 2	Anna, OH	40.4	84.2	VII	70,000	1,2,6,8,9,12
1937 Mar 3	Anna, OH	40.4	84.2	V		1,2,8,11,14

TABLE 2.5-9 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (degrees)	WEST LONGITUDE (degrees)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
1937 Mar 3	Anna, OH	40.4	84.2	III	200	1,8,14
1937 Mar 8	Anna, OH	40.4	84.2	VII-VIII	150,000	1,2,3,6,8,12
1937 Apr 23	Anna, OH	40.4	84.2	III	200	1,8,14
1937 Apr 27	Anna, OH	40.4	84.2	III	200	1,8,14
1937 May 2	Anna, OH	40.4	84.2	IV		1
1937 Jun 29	Peoria, IL	40.7	89.6	II		1
1937 Aug 5	Near St. Louis, MO	38.5	90.2	II-III		1
1937 Aug 5	Granite City, IL	38.7	90.2	II		1
1937 Oct 16	Cincinnati, OH	39.1	84.5	II-III		1
1937 Nov 17	Near Centralia, IL	38.6	89.1	V	8,000	1,2,3,6,12
1938 Feb 12	Porter County, IN	41.6	87.0	V	6,500	1,2
1938 Nov 7	Dubuque, IA	42.5	90.7			1,2
1939 Mar 18	Near Jackson Center, OH	40.4	84.0	II	500	1,8,14
1939 Jun 17	Anna, OH	40.4	84.2	IV	400	1,8,14
1939 July 9	Anna, OH	40.4	84.2	II		1,8,14

TABLE 2.5-9 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (degrees)	WEST LONGITUDE (degrees)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
1939 July 18	Escanaba, MI	45.7	87.1			1
1939 Aug 1	Escanaba, MI	45.7	87.1			1
1939 Nov 7	Escanaba, MI	45.7	87.1	II-III		1
1939 Nov 23	Monroe County, IL	38.2	90.1	V	150,000	1,3
1939 Nov 24	Davenport, IA	41.6	90.6	II-III		1,2
1940 Jan 8	Louisville, KY	38.3	85.8	II-III		1
1940 May 27	Louisville, KY	38.3	85.8	III		1,2
1940 Nov 23	Monroe County, IL	38.2	90.1	VI	150,000	1
1941 Oct 4	St. Louis, MO	38.6	90.2	I		1
1941 Nov 15	Waterloo, IL	38.3	90.2	III		1
1942 Jan	Winfield, MO	39.0	90.7	III		1
1943 Jun 15	House Springs, MO	38.4	90.6	I		1
1943 Jun 18	House Springs, MO	38.4	90.6	I		1
1943 Sept 14	Near St. Louis, MO	38.7	90.3	I		1
1944 Mar 16	Elgin, IL	42.0	88.3	II		1
1944 Sept 25	St. Louis, MO	38.6	90.2	IV	25,000	1

TABLE 2.5-9 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (degrees)	WEST LONGITUDE (degrees)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
1944 Nov 13	Anna, OH	40.4	84.2	III	18,000	1,4,18
1944 Nov 16	Escanaba, MI	45.7	87.1	II		1
1944 Dec 10	Escanaba, MI	45.7	87.1	IV		1
1945 Mar 27	St. Louis, MO	38.6	90.2	II-III		1
1945 May 18	Escanaba, MI	45.7	87.1	II		1
1945 May 21	Near St. Louis, MO	38.7	90.2	III-IV		1
1946 Feb 24	Centralia, IL	38.5	89.1	V	1,500	1,2,10
1946 Nov 7	Washington County, MO	38.0	90.7	II-III		1
1942 Jan 14	St. Louis, MO	38.6	90.2		600	1
1942 Jan 29	St. Louis, MO	38.6	90.2			1
1942 Jan 30	St. Louis, MO	38.6	90.2			1
1942 Mar 1	Kewanee, IL	42.2	89.9	IV-V	3,700	1,2
1942 Nov 17	East St. Louis, IL	38.6	90.2	III-IV	200	1
1942 Dec 27	Maplewood, MO	38.6	90.3	II		1
1943 Feb 9	Marinette County, WI	45.5	88.2	II-III		1

TABLE 2.5-9 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (degrees)	WEST LONGITUDE (degrees)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
1943 Feb 15	Escanaba, MI	45.7	87.1			1
1943 Apr 13	Louisville, KY	38.3	85.8	IV		1
1943 Apr 18	Waterloo, IL	38.3	90.2	I		1
1943 May 20	West Alton, MO	38.9	90.2	I		1
1943 May 24	West Alton, MO	39.9	90.2	I		1
1943 Jun 8	Webster Groves, MO	38.6	90.4	III-IV		1
1947 Mar 16	Kane County, IL	42.1	88.3	IV		1
1947 May 6	Milwaukee, WI	43.0	87.9	IV-V	3,000	1,2
1947 Jun 29	Near St. Louis, MO	38.4	90.2	VI	15,000	1,3
1947 Aug 9	Branch County, MI	42.0	85.0	VI	70,000	1,2,3
1948 Jan 5	Centralia, IL	38.5	89.1	V	300	1,13
1948 Jan 15	Madison County, WI	43.2	89.7	IV-V		1
1948 Apr 20	Iowa City, IA	41.7	91.5	III-IV		1
1949 Jun 8	Ste. Genevieve, MO	38.0	90.1	III	300	1
1949 Aug 11	Clayton, MO	38.7	90.3	II		1
1949 Aug 26	Defiance, MO	38.6	90.8	II-III		1

TABLE 2.5-9 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (degrees)	WEST LONGITUDE (degrees)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
1950 Apr 20	Dayton, OH	39.8	84.2	III		1,8,14
1951 Sept 19	Near Florissant, MO	38.9	90.2	III-IV	1,200	1
1952 Jan 7	Champaign County, IL	40.3	88.3	II-III		1
1953 Sept 11	Near Roxana, IL	38.6	90.1	VI	6,000	1,3
1953 Dec 30	Centralia, IL	38.5	89.1	IV	1,200	1
1954 Aug 9	Petersburg, IN	39.5	87.3	V		1,2
1955 Apr 9	Near Sparta, IL	38.1	89.8	VI	20,000	1,3
1955 May 29	Ewing, IL	38.1	88.9	III-IV		1
1956 Jan 27	Anna, OH	40.4	84.2	V	2,000	1,2,8,14
1956 Mar 13	Fulton County, IL	40.5	90.2	IV	2,000	1
1956 July 18	Oostburg, WI	43.6	87.8	IV		1
1956 Oct 13	Near Milwaukee, WI	42.8	87.9	IV		1
1957 Jan 8	Waupun, WI	43.6	88.7	III-IV		1
1958 Nov 7	Wabash County, IL	38.4	87.9	VI	33,300	1,2,3,9
1959 Jan 6	St. Louis County, MO	38.8	90.4	II-III		1

TABLE 2.5-9 (Cont'd)

DATE	LOCATION	NORTH LATITUDE (degrees)	WEST LONGITUDE (degrees)	MAXIMUM INTENSITY (MM)	FELT AREA (mi^2)	REFERENCES
1967 Feb 2	Lansing, MI	42.7	84.5	IV		1
1967 Aug 5	Jefferson County, MO	38.3	90.6	II		1
1968 Nov 9	Hamilton County, IL	38.0	88.5	VII	585,000	1,2,3,9
1968 Dec 11	Louisville, KY	38.3	85.8	V		1
1971 Feb 12	Wabash County, IL	38.5	87.9	IV	1,300	1
1972 Sept 15	Lee County, IL	41.6	89.4	VI	40,000	1,5
1973 Apr 18	St. Clair County, IL	38.5	90.2	II-III		1
1974 Mar 27	St. Louis, Mo	38.5	90.1			17
1974 Apr 3	Southern Illinois	38.6	88.1	VI		17
1974 Apr 5	Eastern Missouri	38.6	90.9			17
1974 Jun 5	Kentucky	38.6	84.8			17
1974 Jun 5	Southern Illinois	38.6	89.9	V		17
1974 Aug 22	Southern Illinois	38.2	89.7	V		17
1976 Apr 8	Stilesville, IN	39.3	86.8	V		18

TABLE 2.5-9 (Cont'd)

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TABLE 2.5-10

EARTHQUAKES OCCURRING OVER 200 MILES FROM THE SITE FELT AT THE BYRON SITE

DATE	MODIFIED MERCALLI INTENSITY	LOCALITY	EPICENTER LOCATION (degrees) N. LAT.	EPICENTER LOCATION (degrees) W. LONG.	FELT AREA (mi^2)	DISTANCE FROM SITE (mi)
1811 December 16	XI	Northeastern Arkansas Gulf Coast Tectonic Province	35.5	90.5	2,000,000	460
1812 January 23	X-XI	New Madrid, Missouri Gulf Coast Tectonic Province	36.6	89.5	2,000,000	380
1812 February 7	XI-XII	New Madrid, Missouri Gulf Coast Tectonic Province	36.6	89.5	2,000,000	380
1886 August 31	X	Charleston, South Carolina Atlantic Coast Tectonic Province	32.9	80.0	2,000,000	810
1895 October 31	VIII	Charleston, Missouri Gulf Coast Tectonic Province	37.0	89.4	1,000,000	350
1909 September 27	VII	Southern Illinois Central Stable Region	39.0	87.7	30,000	225
1935 November 1	VI	Timiskaming, Canada Laurentian Shield Sub-Province of Central Stable Interior	46.8	79.1	1,000,000	590
1968 November 9	VII	Southern Illinois Central Stable Interior	38.0	88.5	580,000	290

TABLE 2.5-11

RESULTS OF UNCONFINED COMPRESSION TESTS ON ROCK

BORING	FORMATION	DEPTH (ft)	ELEVATION (ft)	DEGREE OF WEATHERING*	ULTIMATE STRENGTH (psi)	POISSON'S RATIO	MODULUS OF ELASTICITY (lb/ft^2)
P-6	Dunleith	23.5	852.0	SW	13,100		
P-13	Dunleith	30.0	846.0	SW	6,400	.23	3.12×10^6
P-16	Dunleith	42.5	831.0	SW	6,500		
P-7	Dunleith	50.5	827.0	MW	2,900		
P-8	Dunleith	51.5	822.5	MW	7,200	.18	3.92×10^6
P-14	Dunleith	71.0	805.0	SW	9,200		
P-6	Dunleith	73.0	802.5	MW	11,600		
P-17	Dunleith	74.0	803.0	SW	6,900		
P-4	Dunleith	98.5	783.0	MW	14,300		
P-5	Dunleith	98.5	774.5	MW	4,300		
G-12	Dunleith	35.0	818.0		17,700		
G-13	Dunleith	36.0	824.5		4,500		
G-3	Dunleith	45.0	810.5		10,300		
G-5	Dunleith	67.0	802.0		7,000		
G-12	Dunleith	70.0	783.0		15,000		
G-13	Dunleith	72.0	788.5		6,000		
P-13	Guttenberg	105.5	770.5	Fr	7,500		
G-4	Guttenberg	67.0	761.5		14,400		
G-12	Guttenberg	80.0	773.0		9,900		
G-13	Guttenberg	91.0	769.5		6,500		
G-5	Guttenberg	93.5	775.5		6,000		

*See Table 2.5-5 Degree of Weathering

TABLE 2.5-11 (Cont'd)

BORING	FORMATION	DEPTH (ft)	ELEVATION (ft)	DEGREE OF WEATHERING*	ULTIMATE STRENGTH (psi)	POISSON'S RATIO	MODULUS OF ELASTICITY (lb/ft^2)
P-16	Quimbys Mill	104.0	769.5	Fr	19,100		
P-6	Quimbys Mill	110.5	765.0	SW	17,700		
G-4	Quimbys Mill	79.0	749.5		7,800		
G-3	Quimbys Mill	86.5	769.0		13,400		
G-12	Quimbys Mill	87.0	766.0		12,300		
G-13	Quimbys Mill	97.0	763.5		7,600		
G-7	Quimbys Mill	107.0	759.0		13,800		
P-16	Nachusa	123.0	750.5	SW	3,600		
P-7	Nachusa	133.0	744.5	SW	6,300		
P-13	Nachusa	133.5	742.5	SW	7,000		
G-4	Nachusa	84.0	744.4		16,400		
G-12	Nachusa	104.0	749.0		19,100		
G-13	Nachusa	112.0	748.5		13,400		
P-13	Grand Detour	147.0	729.0	SW	14,400		
P-7	Grand Detour	149.0	728.5	SW	16,000		
P-6	Grand Detour	149.5	726.0	Fr	3,500		
P-7	Grand Detour	170.0	707.5	MW	5,400		
G-12	Grand Detour	117.5	735.5		12,700		
P-6	Mifflin	176.5	699.0	SW	12,900		
P-13	Mifflin	183.5	692.5	Fr	7,100		
G-12	Mifflin	157.0	696.0		11,100		
P-6	Pecatonica	204.5	671.0	SW	9,700		
P-7	Pecatonica	314.5	663.0	Fr	12,500		

*See Table 2.5-5 Degree of Weathering

TABLE 2.5-12

SUMMARY OF AUGER BORINGS

AUGER HOLE	APPROXIMATE SURFACE ELEVATION[2]	DEPTH TO ROCK (ft)	ELEVATION OF BEDROCK
A-1	870	9.0	861.0
A-2	888	10.0	878.0
A-3	858	9.5	848.5
A-4	840	4.0	836.0
A-5	858	6.0	852.0
A-6	840	12.0	828.0
A-7	806	28.5	777.5
A-8	799	16.5	782.5
A-9	846	13.0	833.0
A-10	836	15.0	821.0
A-11	791	16.5	774.5
A-12	725	39.5	685.5
A-13	776	15.0	761.0
A-14	803	11.5	791.5
A-15	866	7.0	859.0
A-16	863	6.0	857.0
A-17	877	8.5	868.5
A-18	877	7.5	869.5
A-19	870	13.0	857.0

[2]Elevations are in feet and refer to USGS datum.

TABLE 2.5-13

SUMMARY OF PIPELINE AUGER BORINGS AND TEST PITS

AUGER HOLE	SURFACE ELEVATION	STATION NUMBER	DEPTH TO ROCK	ELEVATION OF BEDROCK	DEPTH OF AUGER REFUSAL	ELEVATION OF AUGER REFUSAL
IA-1	808.1	72+50	10.0	798.1	12.5	795.6
IA-2	786.7	22+00	2.0	784.7	7.0	779.7
IA-3	787.1	28+30	2.0	785.1	8.5	778.6
IA-4	770.6	17+30	12.5	758.1	15.5	755.1
IA-5	739.3	14+50	3.0	736.3	5.0	734.3
IA-6	759.1	36+30	14.0	745.1	16.3	742.8
IA-7	869.1	127+10	13.5	855.6	15.0	854.1
IA-8	873.1	121+70	7.0	866.1	9.0	864.1
IA-9	865.5	117+90	6.0	859.5	8.5	857.0
IA-10	864.6	114+00	8.5	856.1	10.5	854.1
IA-11	865.2	105+50	11.0	854.2	14.5	850.7
IA-12	858.6	100+00	19.0	839.6	19.0	839.6
IA-13	854.4	95+00	20.5	833.9	21.0	833.4
IA-14	845.9	84+50	3.0	842.9	5.5	840.4
IA-15	832.5	82+10	5.0	827.5	8.0	824.5
IA-16	829.0	80+00	6.0	823.0	7.5	821.5

TABLE 2.5-13 (Cont'd)

AUGER HOLE	SURFACE ELEVATION	STATION NUMBER	DEPTH TO ROCK	ELEVATION OF BEDROCK	DEPTH OF AUGER REFUSAL	ELEVATION OF AUGER REFUSAL
IA-17	827.9	77+30	17.0	810.9	17.0	810.9
IA-18	823.8	75+00	16.0	807.8	16.0	807.8
IA-19	808.6	70+00	8.0	800.6	8.0	800.6
IA-20	817.3	66+00	7.5	809.8	10.5	806.8
IA-21	830.2	60+00	7.0	823.2	7.0	823.2
IA-22	815.5	55+00	7.5	808.0	9.0	809.5
IA-23	779.4	50+00	6.0	773.4	8.0	771.4
IA-24	776.1	45+50	4.0	772.1	5.5	770.6
IA-25	767.0	43+00	9.5	757.5	11.5	755.5
IA-26	849.8	97+40	8.0	841.8	10.0	839.8
IA-27	857.6	88+30	4.5	853.1	6.0	851.6
IAB-1	861.1	95+80	7.5	853.6	8.5	852.6
IAB-2	861.6	100+80	7.5	854.1	10.0	851.6
IAB-3	865.9	105+80	12.0	853.9	13.0	852.9

TABLE 2.5-13 (Cont'd)

AUGER HOLE	SURFACE ELEVATION	STATION NUMBER	DEPTH TO ROCK	ELEVATION OF BEDROCK	DEPTH OF AUGER REFUSAL	ELEVATION OF AUGER REFUSAL
TEST PIT						
TP-I1 (IA-5)*	739.3	14+50	3.5	735.8	5.0	734.3
TP-I2 (IA-16)*	829.0	80+00	7.0	822.0	7.0	822.0
TP-I3 (IA-21)*	830.2	60+00	6.5	823.7	7.5	822.7

*Identification of Auger Boring Drilled at Test Pit Location

TABLE 2.5-14

SURFACE WAVE DATA

Wave Type:	Sezawa M2
Predominant Particle Motion:	Radial
Predominant Frequency:	16 Hz
Apparent Wavelength:	240 ft
Apparent Velocity:	3730 ft/sec
Observed Length of Wave Train:	3 cycles

TABLE 2.5-15

DOWNHOLE SHEAR WAVE DATA

GEOPHONE DEPTH BORING G-13 (ft)	GEOPHONE DEPTH BORING G-12 (ft)	COMPRESSION (ft/sec)	SHEAR (ft/sec)
75	100	15,500	9,600
75	100	15,500	9,100
75	75	12,000	6,000
75	75	12,000	6,000
150	200	15,500	9,400

TABLE 2.5-16

AMBIENT GROUND MOTION MEASUREMENTS

(September 14, 1972)

AMBIENT STATION	FREQUENCY*, (Hz)	GROUND MOTION**, x 10^{-3}		TRANS	VERT	LONG
1 (Near Boring G-6)	<u>6.5</u>, <u>37.5</u>, <u>50</u>	Displacement	(in.)	.001	--	.001
	<u>11</u>, 50	Acceleration	(in./s/s)	.200	.208	.125
	2, 37.5, 50, 75	Velocity	(in./s)	.185	.150	.100
2 (Near Boring G-14)	<u>4.5</u>, <u>6.5</u>, 8.5, 50	Displacement	(in.)	.0015	.001	.002
	<u>7.5</u>, 50,	Acceleration	(in./s/s)	.250	.083	.100
	<u>7.5</u>, 50	Velocity	(in./s)	.225	.075	.115
3 (Near Boring G-13)	<u>4.5</u>, <u>25</u>, 37.5	Displacement	(in.)	.001	--	--
	<u>37.5</u>	Acceleration	(in./s/s)	.183	.175	.092
	<u>4.5</u>, <u>25</u>, 37.5	Velocity	(in./s)	.125	.060	.050

*Predominant frequencies are underlined.
**Trans = Transverse
Vert = Vertical
Long = Longitudinal

NOTES:

1. Location of ambient station shown on Figure 2.5-63
2. Ambient measurements made by Dames & Moore and Geoterrex, Ltd.

TABLE 2.5-17

SUMMARY OF ESTIMATED STATIC AND DYNAMIC PROPERTIES OF SUBSURFACE MATERIALS AT THE PLANT SITE

		MODULUS OF ELASTICITY			MODULUS OF RIGIDITY (SHEAR MODULUS)								
			DYNAMIC (lb/ft^2)				DYNAMIC (lb/ft^2)				DAMPING FACTOR*		
FORMATION	DEPTH (ft)	STATIC (lb/ft^2)	1.0% STRAIN	0.1% STRAIN	0.1% STRAIN	STATIC (lb/ft^2)	1.0% STRAIN	0.1% STRAIN	0.1% STRAIN	POISSON'S RATIO	1.0% STRAIN	0.1% STRAIN	0.1% STRAIN
Overburden soil	0 to 15	1.0 to $3.0x10^5$	$0.12x10^6$	$0.86x10^6$	$2.50x10^6$	0.3 to $1.0x10^5$	$0.04x10^6$	$0.30x10^6$	$0.87x10^6$	0.44	18	14	7
Dunleith	15 to 90	0.5 to $0.7x10^8$	--	$1.4x10^8$	--	0.2 to $0.3x10^8$	--	$0.5x10^8$	--	0.39	--	1	--
Guttenberg Quimbys Mill	90 to 105	0.7 to $1.0x10^8$	--	$3.8x10^8$	--	0.3 to $0.4x10^8$	--	$1.4x10^8$	--	0.38	--	1	--
Nachusa Grand Detour Mifflin Pecatonica	105 to 227	3.0 to $4.0x10^8$	--	$10.8x10^8$	--	1.3 to $1.7x10^8$	--	$4.5x10^8$	--	0.20	--	1	--

* Expressed as a percent of critical damping.

TABLE 2.5-18

CA-6 BACKFILL MATERIAL GRADATION

SIEVE SIZE	PERCENT PASSING
1-1/2 (in.)	100
1 (in.)	95 ± 5
1/2 (in.)	75 ± 15
No. 4	45 ± 10
No. 16	25 ± 15
No. 200	8 ± 4

TABLE 2.5-19

PLANT LOCATION

RESULTS OF DYNAMIC TRIAXIAL COMPRESSION TESTS

BORING NUMBER	ELEVATION (ft)	SOIL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (lb/ft^3)	CONFINING PRESSURE (lb/ft^2)	CYCLIC DEVIATOR STRESS (lb/ft^2)	SINGLE AMPLITUDE SHEAR STRAIN (%)	MODULUS OF RIGIDITY (lb/ft^2)	DAMPING (%)
P-1	874.0	SC (till)	18.6	110	500	133.1	0.005767	1.15×10^6	
						187.8	0.010573	$.89 \times 10^6$	
						281.8	0.020425	$.69 \times 10^6$	
						414.8	0.039549	$.32 \times 10^6$	
						532.2	0.074492	$.36 \times 10^6$	
						673.1	0.151386	$.22 \times 10^6$	
						814.0	0.302773	$.13 \times 10^6$	
						939.2	0.600739	$.08 \times 10^6$	
						1095.7	1.261553	$.04 \times 10^6$	
P-15	871.5	SC (till)	19.1	109	500	117.4	0.005532	1.06×10^6	
						242.6	0.020745	$.58 \times 10^6$	
						344.4	0.039184	$.44 \times 10^6$	
						469.6	0.071454	$.33 \times 10^6$	
						641.8	0.142908	$.22 \times 10^6$	
						782.7	0.285816	$.14 \times 10^6$	
						860.9	0.599291	$.07 \times 10^6$	
						860.9	1.210106	$.04 \times 10^6$	
P-22	864.5	SC (till)	16.6	113	750	133.1	0.005219	1.27×10^6	
						187.8	0.009964	$.94 \times 10^6$	
						281.8	0.020164	$.70 \times 10^6$	
						391.3	0.039142	$.50 \times 10^6$	
						500.9	0.073540	$.34 \times 10^6$	

TABLE 2.5-19 (Cont'd)

BORING NUMBER	ELEVATION (ft)	SOIL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (lb/ft^3)	CONFINING PRESSURE (lb/ft^2)	CYCLIC DEVIATOR STRESS (lb/ft^2)	SINGLE AMPLITUDE SHEAR STRAIN (%)	MODULUS OF RIGIDITY (lb/ft^2)	DAMPING (%)
						563.5	0.147080	$.19 \times 10^6$	
						626.1	0.294161	$.11 \times 10^6$	
						626.1	0.593066	$.05 \times 10^6$	
P-16	862.5	SC	9.8	132	1000	133.1	0.004787	1.39×10^6	9.4
		(till)				230.9	0.010293	1.12×10^6	8.1
						360.0	0.018551	$.97 \times 10^6$	9.0
						532.2	0.039495	$.67 \times 10^6$	11.1
						704.4	0.074202	$.47 \times 10^6$	13.4
						829.6	0.143617	$.29 \times 10^6$	15.7
						907.9	0.287234	$.16 \times 10^6$	16.4
						860.9	0.574468	$.07 \times 10^6$	16.1
						1174.0	1.196808	$.05 \times 10^6$	15.6

TABLE 2.5-20

RIVER SCREEN HOUSE
RESULTS OF DYNAMIC TRIAXIAL COMPRESSION TESTS

BORING NUMBER	ELEVATION (ft)	SOIL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (lb/ft^3)	CONFINING PRESSURE (lb/ft^2)	CYCLIC DEVIATOR STRESS (lb/ft^2)	SINGLE AMPLITUDE SHEAR STRAIN (%)	MODULUS OF RIGIDITY (lb/ft^2)	DAMPING (%)
G-23*	622.5	SW (alluvium)	16.8	115	7000	538	0.0022	12.20×10^6	---
						871	0.0135	3.20×10^6	2.4
						1444	0.0386	1.87×10^6	5.8
						1805	0.0555	1.63×10^6	7.4
						2439	0.0911	1.34×10^6	9.7
						2914	0.1128	1.29×10^6	9.3
						3358	0.1436	1.17×10^6	15.1
						4941	0.3164	0.78×10^6	17.3
						6557	0.4858	0.67×10^6	17.6
						7918	0.6258	0.63×10^6	16.6
						10611	0.9884	0.53×10^6	10.5

TABLE 2.5-20 (Cont'd)

BORING NUMBER	ELEVATION (ft)	SOIL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (lb/ft^3)	CONFINING PRESSURE (lb/ft^2)	CYCLIC DEVIATOR STRESS (lb/ft^2)	SINGLE AMPLITUDE SHEAR STRAIN (%)	MODULUS OF RIGIDITY (lb/ft^2)	DAMPING (%)
G-23*	572.5	SW (alluvium)	17.0	115	3500	1267	0.0054	11.72×10^6	---
						1539	0.0177	4.35×10^6	14.0
						2122	0.0533	1.99×10^6	15.6
						2565	0.0835	1.94×10^6	21.4
						2945	0.1152	1.28×10^6	18.6
						5954	0.2765	1.08×10^6	20.7
						9502	0.6020	0.78×10^6	20.5
						12353	0.8786	0.70×10^6	25.2
RS-4	652.5	SP (alluvium)	10.7	131	1600	245	0.0122	10.04×10^5	9.9
						319	0.0194	8.25×10^5	12.2
						499	0.0504	4.96×10^5	14.8
						644	0.0846	3.81×10^5	15.7
						725	0.1139	3.19×10^5	16.0
						815	0.1497	2.73×10^5	13.6
						960	0.3156	1.52×10^5	16.2
						1002	0.6019	0.83×10^5	17.1
						1257	0.8947	0.70×10^5	18.0

*Reconstituted Sample

TABLE 2.5-20 (Cont'd)

BORING NUMBER	ELEVATION (ft)	SOIL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (lb/ft^3)	CONFINING PRESSURE (lb/ft^2)	CYCLIC DEVIATOR STRESS (lb/ft^2)	SINGLE AMPLITUDE SHEAR STRAIN (%)	MODULUS OF RIGIDITY (lb/ft^2)	DAMPING (%)
RS-3	647	SW (alluvium)	16.7	116	2400	224	0.0078	14.48×10^5	4.5
						514	0.0242	10.63×10^5	7.0
						821	0.0476	8.62×10^5	9.1
						1052	0.0740	7.12×10^5	12.3
						1280	0.1026	6.24×10^5	12.9
						1456	0.1237	5.89×10^5	13.5
						1760	0.2605	3.38×10^5	16.9
						1857	0.5185	1.79×10^5	18.5
						2053	0.7632	1.34×10^5	17.7
						2356	1.0396	1.13×10^5	15.8

TABLE 2.5-21

SHOCKSCOPE TEST DATA

BORING	FEET	FORMATION	DENSITY (lb/ft^3)	COMPRESSIONAL WAVE VELOCITY (ft/sec)
G-12	20	Dunleith	163.9	18,100
G-12	62.5	Dunleith	163.7	17,500
G-12	79	Guttenberg	162.7 (Vuggy)	15,600
G-12	134	Grand Detour	154.0	12,500
G-12	154	Mifflin	167.1	19,500
G-6	281	St. Peter	130.0	5,500
G-6	293	St. Peter	132.0	6,500
G-6	303	St. Peter	132.0	6,500

TABLE 2.5-22

RESONANT COLUMN TEST DATA

BORING NUMBER	ELEVATION (ft)	FORMATION ROCK TYPE	DENSITY (lb/ft^3)	CONFINING PRESSURE (lb/ft^2)	SHEAR STRAIN AMPLITUDE (%)	SHEAR WAVE VELOCITY (ft/sec)	MODULUS OF RIGIDITY (lb/ft^2)	DAMPING (%)
P-3	805.0	Dunleith Dolomite	160.6	3600	0.00049	3014	$45.54x10^6$	4.85 (ave)
				7200	0.00045	3128	$49.05x10^6$	
				14832	0.00042	3260	$53.27x10^6$	
P-2	793.5	Dunleith Dolomite	165.3	3600	0.00036	3165	$51.67x10^6$	5.2
				7200	0.00051	3650	$68.75x10^6$	2.9
P-3	769.5	Guttenberg Dolomite	165.1	3600	0.00054	3033	$47.41x10^6$	4.65 (ave)
				7200	0.00048	3215	$53.25x10^6$	
				14832	0.00046	3422	$60.33x10^6$	
P-2	759.5	Nachusa Dolomite	159.7	4320	0.00030	4341	$93.88x10^6$	5.0 (ave)
				8640	0.00023	4561	$103.62x10^6$	
				14976	0.00020	4828	$116.08x10^6$	

TABLE 2.5-22 (Cont'd)

BORING NUMBER	ELEVATION (ft)	FORMATION ROCK TYPE	MOISTURE CONTENT (%)	DENSITY (lb/ft^3)	CONFINING PRESSURE (lb/ft^2)	SHEAR STRAIN AMPLITUDE (%)	SHEAR WAVE VELOCITY (ft/sec)	MODULUS OF RIGIDITY (lb/ft^2)	DAMPING (%)
G-23	661.3	Alluvium	10.0	119	749	.00025	498	$10.05x10^5$	1.1
					1498	.00064	678	$18.64x10^5$	1.7
					2246	.00056	740	$22.20x10^5$	1.6
G-23	620.8	Alluvium	7.7	139	2995	.00010	875	$35.74x10^5$	1.3
					4003	.00003	1039	$50.36x10^5$	2.6
					4996	.00003	1108	$57.35x10^5$	2.3
G-23	570.8	Alluvium	12.8	121	5990	.00014	1063	$47.73x10^5$	2.0
					6854	.00025	1153	$56.14x10^5$	2.3
					8006	.00023	1200	$60.87x10^5$	2.2

TABLE 2.5-23

IN SITU DENSITIES

ELEVATION (ft)	DRY DENSITY (lb/ft^3)	WET DENSITY (lb/ft^3)
679-649	120	135
649-634	125	135
634-624	116	132
624-565	125	135

TABLE 2.5-24

SUMMARY OF LIQUEFACTION TESTS

BORING NUMBER	ELEVATION (DEPTH) (ft)	SOIL TYPE	GRAIN SIZE D_{50} mm	DRY DENSITY (lb/ft^3)	FINAL WATER CONTENT (%)	EFFECTIVE CONFINING PRESSURE σc(lb/ft^2)	CYCLIC VERTICAL STRESS $\Delta\sigma_v$(lb/ft^2)	STRESS RATIO $\frac{\Delta\sigma_v}{2\sigma_c}$	CYCLES TO LIQUEFACTION N
RS 1	652.2 (24.0)	Gravelly Sand SW	2.0	118	10.1	1600	913	0.286	19
RS 1	647.2 (29.0)	Gravelly Sand SW	1.6	130	10.2	2400	1275	0.265	62
RS 1	632.2 (44.0)	Sand SW	0.8	118	14.3	3400	2731	0.402	31
RS 1	627.2 (49.0)	Sand SP	0.4	113	19.5	3400	1500	0.220	220
RS 2	626.7 (49.0)	Gravelly Sand SW	0.8	127	13.0	3400	2040	0.300	60
RS 2	621.7 (54.0)	Gravelly Sand SW	0.6	120	14.6	3400	1206	0.170	>1,000
RS 3	657.2 (18.8)	Medium Sand SP	0.5	107	19.5	1600	909	0.284	19

TABLE 2.5-24 (Cont'd)

BORING NUMBER	ELEVATION (DEPTH) (ft)	SOIL TYPE	GRAIN SIZE D_{50} mm	DRY DENSITY (lb/ft^3)	FINAL WATER CONTENT (%)	EFFECTIVE CONFINING PRESSURE $\sigma c(lb/ft^2)$	CYCLIC VERTICAL STRESS $\Delta\sigma_v(lb/ft^2)$	STRESS RATIO $\frac{\Delta\sigma_v}{2\sigma_c}$	CYCLES TO LIQUEFACTION N
RS 3	642.2 (33.8)	Gravelly Sand SW	1.2	130	11.7	2400	1487	0.310	37
RS 4	657.2 (19.1)	Gravelly Sand SW	---	126	11.2	1600	1015	0.317	88
RS 4	647.2 (29.1)	Gravelly Sand SW	1.0	123	13.5	2400	706	0.147	201
RS 4	622.2 (54.1)	Medium Sand SP	0.7	110	19.2	3400	1770	0.260	14

TABLE 2.5-25

SUMMARY OF LIQUEFACTION ANALYSES

(Artificial Base Rock Motion)

DEPTH (ft)	EFFECTIVE OVERBURDEN PRESSURE σ_c (kips/ft^2)	CRITICAL NUMBER OF CYCLES N_c		AVERAGE CYCLIC SHEAR STRESS FOR N_c CYCLES (kips/ft^2) τ_{avg}		AVERAGE CYCLIC SHEAR STRESS CAUSING LIQUEFACTION IN N_c CYCLES (kips/ft^2) τ_{liq}		MINIMUM FACTOR OF SAFETY WITH RESPECT TO LIQUEFACTION FOR N_c CYCLES, τ_{liq}/τ_{avg}	
		Case 1	Case 2	Case 1	Case 2	Case 1	Case 2	Case 1	Case 2
8	0.58	5	6	0.088	0.146	0.207	0.196	2.35	1.34
16	1.16	5	4	0.164	0.305	0.415	0.439	2.53	1.44
24	1.74	4	4	0.239	0.426	0.656	0.656	2.74	1.54
32	2.32	3	4	0.304	0.529	0.950	0.873	3.13	1.65
40	2.90	3	4	0.338	0.610	1.188	1.098	3.51	1.80
48	3.48	3	4	0.368	0.676	1.425	1.311	3.87	1.94
56	4.06	3	4	0.392	0.729	1.663	1.531	4.24	2.10
64	4.64	3	4	0.413	0.774	1.900	1.757	4.60	2.27
72	5.22	3	4	0.434	0.814	2.138	1.986	4.93	2.44
80	5.80	5	4	0.405	0.854	2.074	2.186	5.12	2.56

TABLE 2.5-26

SUMMARY OF LIQUEFACTION ANALYSES*

(Base Rock Motions from Actual Earthquake Histories)

DEPTH (ft)	EFFECTIVE OVERBURDEN PRESSURE σ_c (kips/ft^2)	CRITICAL NUMBER OF CYCLES N_c No. 1 EQK**	NO.2 EQK***	AVERAGE CYCLIC SHEAR STRESS FOR N_c CYCLES (kips/ft^2) τ_{avg} No. 1 EQK	NO. 2 EQK	AVERAGE CYCLIC SHEAR STRESS CAUSING LIQUEFACTION IN N_c CYCLES (kips/ft^2) τ_{liq} No. 1 EQK	No. 2 EQK	MINIMUM FACTOR OF SAFETY WITH RESPECT TO LIQUEFACTION FOR N_c CYCLES, τ_{liq}/τ_{avg} No. 1 EQK 1	No. 2 EQK
13.0	0.94	2	4	0.293	0.253	0.413	0.358	1.41	1.41
22.0	1.60	2	4	0.376	0.349	0.702	0.604	1.87	1.73
30.0	2.18	2	4	0.413	0.416	0.956	0.822	2.31	1.98
45.0	3.27	2	2	0.491	0.569	1.433	1.433	2.92	2.52
55.0	3.96	2	2	0.527	0.598	1.737	1.737	3.30	2.91

* All analyses were made using the higher bound of the dynamic soil properties.
** No. 1 EQK: The S80E component of the March 22, 1957 Golden Gate Park earthquake.
*** No. 2 EQK: The N-S component of the 1935 Helena, Montana earthquake.

TABLE 2.5-27

FOUNDATION DATA

SAFETY CATEGORY I	PLAN DIMENSIONS (ft x ft)	FOUNDATION ELEVATION* (ft)	STRATIGRAPHIC UNIT AT FOUNDATION ELEVATION	STATIC APPLIED FOUNDATION LOAD (lb/ft^2)	EXTREME** SEISMIC FOUNDATION LOAD (lb/ft^2)
Reactor containment	157 (Diameter)	831 (core at 809)	Dunleith Formation	6,000 to 10,000	21,000
Auxiliary (T-Shaped)	80 x 422 87 x 92	792 and 838	Dunleith Formation	5,000 to 10,000	35,000
Fuel handling	92 x 127	848 and 864	Dunleith Formation	4,000 to 5,000	15,000
Essential service water area	105 x 123	792	Dunleith Formation	5,000	10,000
Essential service water cooling tower	45 x 360	865	Dunleith Formation	3,000	6,000
River screen house	72 x 119	661 and 666.5	Alluvium	2,000	4,500

TABLE 2.5-27 (Cont'd)

OTHER MAJOR STRUCTURES	APPROXIMATE PLAN DIMENSIONS (ft x ft)	FOUNDATION ELEVATION* (ft)	STRATIGRAPHIC UNIT AT FOUNDATION ELEVATION	STATIC APPLIED FOUNDATION LOAD (lb/ft^2)	EXTREME SEISMIC FOUNDATION LOAD (lb/ft^2)
Turbine building	130 x 735	820 to 866	Dunleith Formation Controlled Compacted Crushed Rock	4,000	----
Turbine generator pedestal	60 x 210	831	Dunleith Formation	6,000	----
Radwaste building	90 x 145	863	Controlled Compacted Crushed Rock	2,000	----
Service building	130 x 145	867	Controlled Compacted Crushed Rock	2,000	----
Heater bay	50 x 500	867	Dunleith Formation and Controlled Compacted Crushed Rock	4,000	----
Pump house	95 x 195	844 and 848	Dunleith Formation	3,000	----

* Elevation of the bottom of the foundation to the nearest foot, U.S.G.S. datum.
**Represents foundation loads during SSE event.

TABLE 2.5-28

RIVER SCREEN HOUSE SUBGRADE

IN-PLACE DENSITY TESTS*

TEST NUMBER	NORTH COORDINATE	WEST COORDINATE	USGS ELEVATION (ft)	MAXIMUM DRY DENSITY (pcf)	FIELD DRY DENSITY (pcf)	FIELD MOISTURE CONTENT (%)	PERCENT COMPACTION	UNIFIED SOIL CLASSIFICATION & COMMENTS
805	70+79	81+13	669.0	-	142.3	5.5	-	GW-Before Compaction
806	70+76	81+24	662.0	108.5	101.1	5.3	93.2	SP-Before Compaction
807	70+80	81+88	662.0	-	133.0	3.8	-	GW-Before Compaction
809	71+16	81+45	662.0	108.5	107.9	4.0	99.4	SP-After Compaction
810	70+64	81+41	662.0	-	145.2	3.6	-	GP-After Compaction

* Modified Proctor (ASTM D-1557) of fine sand pockets at river screen house - maximum dry density = 108.5 pcf, optimum moisture content = 8.9%

TABLE 2.5-29

GRAIN SIZE ANALYSIS

(ASTM 422-63)

TEST NO.	PERCENT PASSING 3" SIEVE	2" SIEVE	1-1/2" SIEVE	1" SIEVE	3/4" SIEVE	1/2" SIEVE	3/8" SIEVE	#4 SIEVE	#8 SIEVE	#16 SIEVE	#30 SIEVE	#40 SIEVE	#50 SIEVE	#100 SIEVE	#200 SIEVE	UNIFIED SOIL CLASSI-FICATION
805*	-	97.0	92.6	87.1	-	66.0	-	37.3	-	19.1	-	-	-	-	0.5	GW
806	-	-	-	-	-	-	-	99.9	99.7	99.5	99.0	-	48.2	1.3	0.5	SP
810	94.1	79.0	-	57.5	-	57.3	-	38.6	-	25.8	-	-	-	-	3.9	GP
123	-	100.0	-	90.6	-	73.2	-	46.6	-	22.9	-	-	-	-	0.9	GW
820**	-	-	100.0	99.5	-	82.5	-	54.2	-	36.4	-	-	-	-	8.1	SP-SM
ST-4***	-	-	-	-	100.0	-	89.6	79.5	73.8	70.7	64.4	54.6	43.3	26.0	17.9	SC

* Samples for Tests #805, 806, 810, & 123 taken from river screen house subgrade.
** Test 820 is (CA 6 - 10) backfill used at river screen house.
*** Test ST-4 is excavated material used as backfill at river screen house.

TABLE 2.5-30

RIVER SCREEN HOUSE BACKFILL

IN-PLACE DENSITY TESTS*

TEST NUMBER	NORTH COORDINATE**	WEST COORDINATE**	USGS ELEVATION (ft)	MAXIMUM DRY DENSITY (pcf)	FIELD DRY DENSITY (pcf)	FIELD MOISTURE CONTENT (%)	PERCENT COMPACTION	RETEST OF TEST NUMBER
3-2	70+42	80+96	669.0	138.2	140.0	6.6	100+	-
3-3	70+43	81+11	671.0	138.2	120.4	5.1	87.1	-***
10-1	71+26	81+31	668.3	138.2	137.0	9.3	99.1	-
10-2	71+20	81+71	668.8	138.2	128.0	10.5	92.5	-
10-3	71+21	81+31	669.3	138.2	136.7	7.6	98.9	-
10-4	71+20	81+71	668.8	138.2	140.5	9.5	100+	10-2
15-1	71+21	81+31	669.9	138.2	140.9	6.8	100+	-
15-2	70+95	80+78	672.5	138.2	120.9	7.5	87.5	-
15-3	71+20	81+31	672.8	138.2	159.4	6.0	100+	-
16-1	70+35	81+22	674.5	138.2	141.9	6.2	100+	-
16-2	70+76	80+73	672.8	138.2	144.1	5.6	100+	15-2
19-1	70+38	80+87	676.5	136.8	138.5	7.4	100+	-
19-2	70+40	81+82	677.3	136.8	141.5	8.3	100+	-
20-1	71+20	81+02	675.5	136.8	134.4	7.8	98.3	-

* Modified Proctor (ASTM D-1557-70) used for river screen house backfill (see tabulation below):

** Coordinates approximated from test location description.

*** No retest performed.

TEST NUMBER	MAXIMUM DRY DENSITY (pct)	OPTIMUM MOISTURE CONTENT (%)	MATERIAL
ST-4	138.2	6.5	SC
820	136.8	8.7	SP-SM (CS 6-10)

TABLE 2.5-31

SETTLEMENT RECORDS, RIVER SCREEN HOUSE

SETTLEMENT MONUMENT	DATE OF INITIAL READING	INITIAL ELEVATION (ft)	DATE OF LAST READING	ELEVATION OF LAST READING	MEASURED SETTLEMENT (in)
SBM 1	8-22-77	663.49	7-21-78	663.48	0.12
SBM 2	8-22-77	663.52	7-21-78	663.49	0.36
SBM 3	8-22-77	663.50	7-21-78	663.48	0.24
SBM 4	8-25-77	669.04	7-21-78	669.02	0.24
SBM 5	12-7-77	686.47	1-22-79	686.47	0.00
SBM 6	12-7-77	686.49	1-22-79	686.50	0.00
SBM 7	12-7-77	686.49	1-22-79	686.50	0.00

TABLE 2.5-32

RESULTS OF DENSITY TEST

MATERIAL NO.	MAXIMUM DRY DENSITY, PCF ASTM D-1557	MAXIMUM/MINIMUM[3] DRY DENSITY, PCF ASTM-D2049	95 PERCENT OF MODIFIED PROCTOR EXPRESSED AS RELATIVE DENSITY (%)
1[1]	140.6	136.0/101.8	95
2[1]	140.8	135.5/102.4	96
3[2]	147.0	139.2/105.1	101
4[2]	142.6	131.9/105.3	110

[1]Tests on CCFI Material, Marble Hill Nuclear Power Plant.

[2]Report "Field Density and Laboratory Investigation of the Crushed Stone Fill, Callaway Plant Units 1 and 2." Dames & Moore Report for Union Electric Company dated August 8, 1975.

[3]Wet Method.

TABLE 2.5-33

SOIL LAYERS AND MATERIAL PROPERTIES

LAYER	THICKNESS FEET	DEPTH[3] FEET	UNIT WEIGHT[4] PCF	EFFECTIVE OVERBURDEN PSF
1	5.0	2.5	135	340
2	8.0	9.0	135	970
3	9.0	17.5	135	1,580
4	8.0	26.0	135	2,220
5	8.0	34.0	135	2,780
6	7.0	41.5	135	3,320
7	10.0	50.0	135	3,940
8	10.0	60.0	135	4,670
9	10.0	70.0	135	5,390
10	10.0	80.0	135	6,120
11	10.0	90.0	135	6,850
12	10.0	100.0	135	7,570
13	10.0	110.0	135	8,300
14	Rock			

[3]Depth of middle of layer

[4]Groundwater table was assumed to be at 5 feet.

TABLE 2.5-34

SHEAR MODULUS FOR SOILS ALONG THE ESWS PIPELINE

DEPTH FEET	EFFECTIVE OVERBURDEN PRESSURE PSF	MEAN EFFECTIVE PRESSURE PSF	DYNAMIC SHEAR MODULUS Gmax, KSF
Upland Section			
5	650	430	1,330
10	1,300	870	1,880
15	1,850	1,230	2,240
20	2,400	1,600	2,560
25	2,950	1,970	2,840
30	3,500	2,330	3,090
35	4,050	2,700	3,320
40	4,600	3,070	3,540
Flood Plain Section			
5	670	450	1,910
10	1,040	690	2,360
15	1,400	930	2,740
20	1,760	1,180	3,090
25	2,130	1,420	3,390
30	2,490	1,660	3,670
35	2,850	1,900	3,920
40	3,220	2,150	4,170
45	3,580	2,400	4,410
50	3,940	2,630	4,610
60	4,670	3,110	5,020
70	5,390	3,600	5,400
80	6,120	4,080	5,750
90	6,850	4,570	6,080
100	7,570	5,050	6,400
110	8,300	5,530	6,700
115	8,660	5,770	6,840

TABLE 2.5-35

GEOPHYSICAL PROPERTIES AND NORMALIZED SHEAR MODULUS FACTOR FOR GRANULAR MATERIAL

SITE	SOIL CONDITIONS	MEAN BLOW COUNT SPT	DEPTH (ft)	SHEAR WAVE VELOCITY (fps)	DEPTH TO WATER TABLE (ft)	K_0*	K_2
Cholame-Shandon Array California (In situ Impulse/ Downhole, 1975)	SM Alluvium (Holocene)	81	135	900	26	1.0	31
		56	165	1,100		1.0	43
Terminal Substation El Centro, California (Downhole, 1975)	SM Lake Deposit (Quatenary)	88	95	850	33	1.0	31
Highway Test Lab Olympia, Washington (Crosshole, 1974)	SP-SM Glaciolacustrine Deposit	29	25	750	12	0.6	53
CIT Millikan Library Pasadena, California (Downhole, 1975)	SM Alluvium (Pleistocene)	126	40	1,550	238	1.0	132
		180	80	1,550		1.0	93
		155/5"	120	1,950		1.0	121
4800 Oak Grove Pasadena, California (Downhole, 1978)	SW-GW	42	20	1,100	225	0.8	103
	SM-SW	103	40	1,600		1.0	143
	SM-SW	(very dense)	80	1,600		1.0	101
	SM-SW-GW Alluvium (Pleistocene)	(very dense)	140	2,000		1.0	120

*Estimated based on Gibbs and Holtz, 1957 (Reference 8); Marcuson and Bieganousky, 1976 (Reference 10); and Mayne and Kulhawy, 1982 (Reference 9).

TABLE 2.5-35 (Cont'd)

SITE	SOIL CONDITIONS	MEAN BLOW COUNT SPT	DEPTH (ft)	SHEAR WAVE VELOCITY (fps)	DEPTH TO WATER TABLE (ft)	K_0*	K_2
State Building	SP	36	25	1,000	20	0.6	87
San Francisco, California	(Quaternary)	49	55	1,100		1.0	70
(Downhole, 1978)	Sediments)	122	80	1,600		1.0	127
Lincoln School Tunnel	SM	71	25	1,200	<200	1.0	98
Taft, California	Alluvium	121	50	1,200		1.0	69
(Downhole, 1976)	(Quaternary)	103	80	1,600		1.0	97
Noranda Aluminum Plant	SP/SP-SM	23	25	850	11	0.4	77
New Madrid, Missouri	Alluvium	32	75	900		0.9	45
(Downhole, 1979)	(Quaternary)	69	120	1,000		1.0	43
MSU Roberts Hall	GW	35	14	750	8	1.0	60
Bozeman, Montana	Alluvium	85/6"	25	1,300		1.0	146
(In situ Impulse, 1976)	(Quaternary)						
PSU Cramer Hall	GW	106/6"	105	1,800	133	1.0	112
Portland, Oregon							
(Downhole, 1978)							
USU Old Main Building	SW-GW	23	15	900	150	0.6	86
Logan, Utah	Alluvium	-	50	1,300		0.5	103
(Downhole, 1976)	(Quaternary)	66	90	1,600		1.0	96
1900 Avenue of the Stars	SP/SM	102/6"	80	1,300	64	1.0	70
Los Angeles, California	Pleistocene	124	120	1,800		1.0	119
(Downhole, 1975)	(Some Cementation)						
Hollywood Storage Bldg.	SM w/Gravel	61	120	1,400	40	1.0	77
Los Angeles, California	Alluvium						
(Downhole, 1979)	(Quaternary)						

Attachments 2.5A through 2.5J have been deleted intentionally.